Design & Reinforcement of Frames.

نسألكم الدعاء

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اذا حملت تطبيق RC Structures YASSER EL LEATHY على تليفونك المحمول او اللوح السطحى ستستطيع أن تشغل أفلام شرح للمقاطع التى تحتوى على رمز

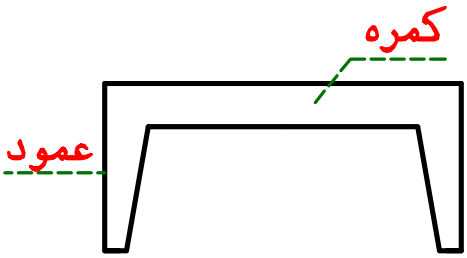
Design & Reinforcement of Frames. Table of Contents.

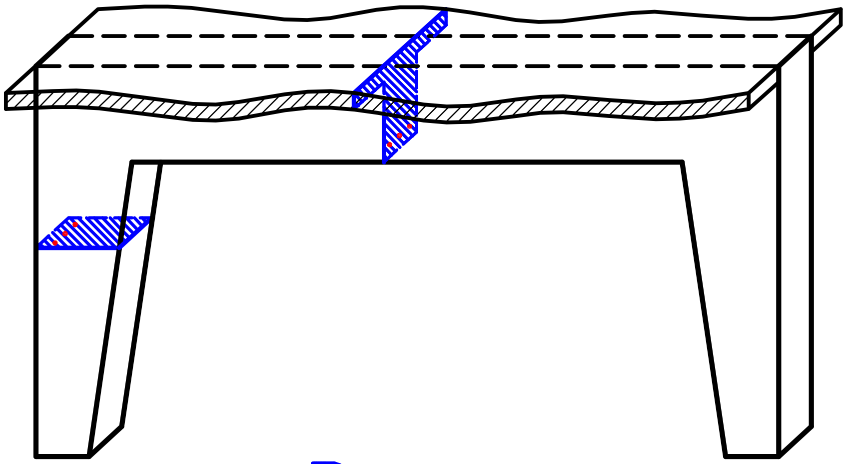
Design of Frames Sections.

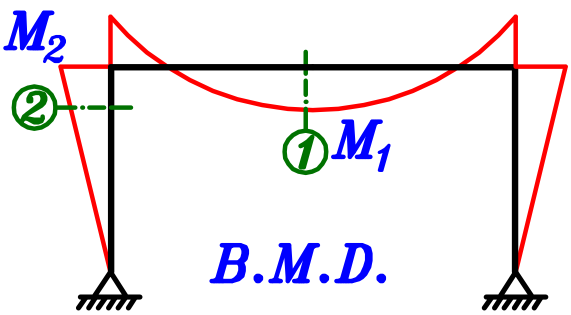
Design of Sections Subjected to M, P

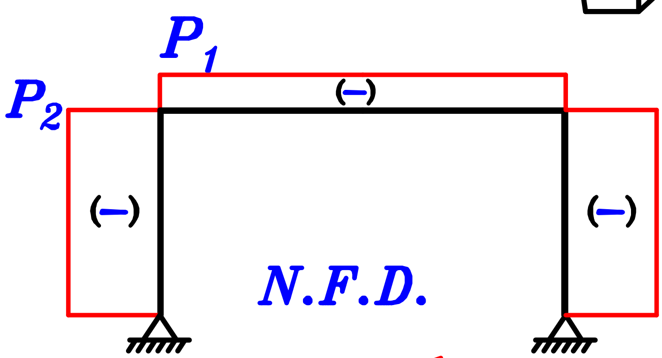
تتعرض القطاعات فى ال **Frames** إلى **Moment & Normal**

و ممكن أن تكون هذه القطاعات إما فى كمرات أو فى أعمده .

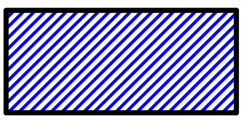








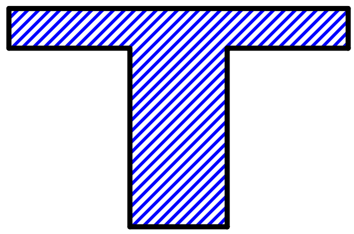
عمود



Sec. (2−2)

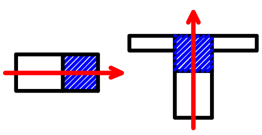
Designed on M_2 , P_2

كمره



Sec. (1−1)

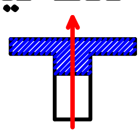
Designed on M_1 , P_1



فى حاله وجود M , P يجب عمل تصميم للقطاع على أنه **R−Sec.**

أما فى حاله وجود M فقط فيجب مراعاه إذا ما كان القطاع

R−Sec. or **T−Sec.** or **L−Sec.**



Steps of Design :

1 – Get Dimensions of the section. $(b \times t)$

2 – Check IF P neglected or not.

3 – Get Reinforcement A_s , A_s'

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1 – Get Dimensions of the section. $(b \times t)$

Take $b = (300\ mm$ or $350\ mm$ or $400\ mm)$

To get t get the bigger value of t_1 (Bending) , t_2 (Normal)

– Get $d_1 = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}}$ take $C_1 = 3.5$, $J = 0.78$ (as R–Sec.)

$t_1 = d_1 + cover$ where $cover = 50\ mm$ IF $t \leqslant 1000\ mm$

$= 100\ mm$ IF $t > 1000\ mm$

– Get $t_2 \xrightarrow{Take} \mu = \dfrac{A_s}{b\,t_2} = 1.0\ \% \longrightarrow A_s = \dfrac{b\,t_2}{100}$

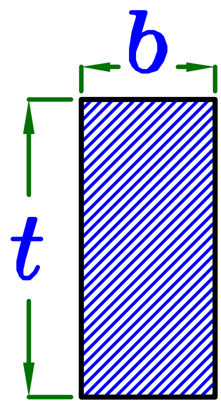
From $P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$

$$\therefore P_{U.L.} = 0.35\, b\, t_2\, F_{cu} + 0.67\, \frac{b\, t_2}{100}\, F_y$$

$$\therefore \boxed{P_{U.L.} = \left(0.35\, b\, F_{cu} + 0.67\, \frac{b}{100}\, F_y\right) t_2}$$

– t_o = The bigger value of t_1 & t_2

– $t = (1.1 \rightarrow 1.3)\, t_o$



2- Check:

✓✓ 1- IF $K = \frac{P_{U.L.}}{F_{cu}\, b\, t} \leqslant 0.04 \longrightarrow$ neglect $P_{U.L.}$

and Design the Sec. on B.M. only as Beams.

$\therefore\ d = d_1 = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}}$ take $C_1 = 3.5$, $J = 0.78$ (R-Sec.)

take $C_1 = 6.0$, $J = 0.826$ (T-Sec. , L-Sec.)

ملحوظه هامه :

فى بدايه التصميم نعمل تصميم على M, P على أن القطاع R-sec.

و لكن اذا أهملنا ال P فنعمل تصميم على M فقط فيجب مراعاه

اذا كان القطاع R-sec. or T-sec.

Get $\boxed{e = \frac{M_{U.L.}}{P_{U.L.}}}$

IF $\frac{e}{t} \leqslant 0.05 \longrightarrow$ neglect $M_{U.L.}$

and Design the Sec. on N.F. only as Columns.

$P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$ Take $\mu = 1.0\ \%$

$\therefore\ P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, \frac{A_c}{100}\, F_y$

Get A_c , A_s

ممكن إهمال هذه الخطوه

IF $K = \frac{P_{U.L.}}{F_{cu}\, b\, t} > 0.04$ Design the Sec. on both B.M. & N.F.

3 - Get Reinforcement $A_s , A_s`$

لحساب كميه الحديد طريقتين : ١ - طريقه دقيقه (صعبه)

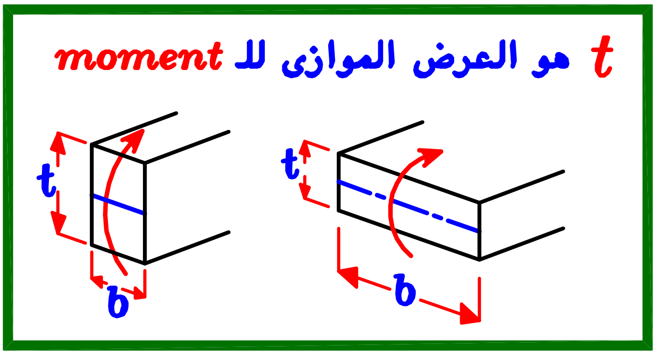
٢ - طريقه تقريبيه (المعمول بها فى هذا الملف)

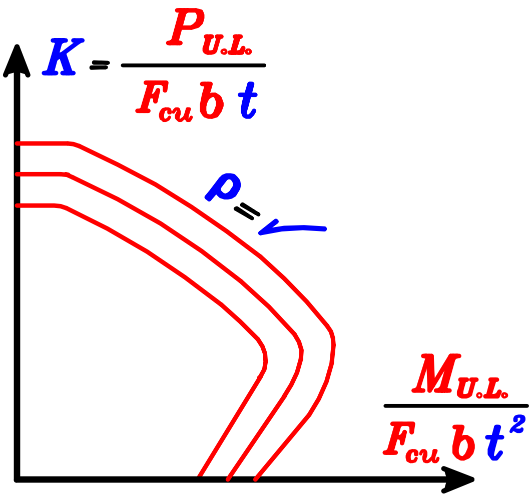
1- Exact Method. ١ - طريقه دقيقه (صعبه)

Use Interaction Diagram	ECCS Page (4-20) → (4-63)

Interaction Diagram. (I.D.)

t هو العرض الموازى لل moment





لتحديد الصفحه المطلوبه نحدد ثلاثه قيم . F_y , α , ζ

Chart Key مفتاح الجدول

يوجد فى كل صفحه من صفحات ال I.D. فى الجداول مفتاح للجدول لتحديد أى جدول سوف نستخدمه

Chart Key

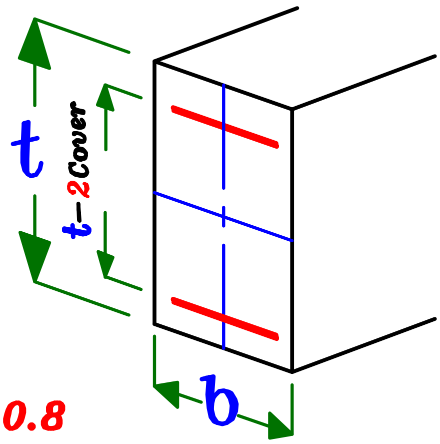
$F_y = \checkmark$
$\zeta = \checkmark$
$\alpha = \frac{A_s`}{A_s} = 1$

- F_y = Type of Steel
  - 240
  - 280
  - 360 ✓✓
  - 400 ✓✓

- $\alpha = \frac{A_s`}{A_s}$
  - 0.8
  - 1.0 ✓✓

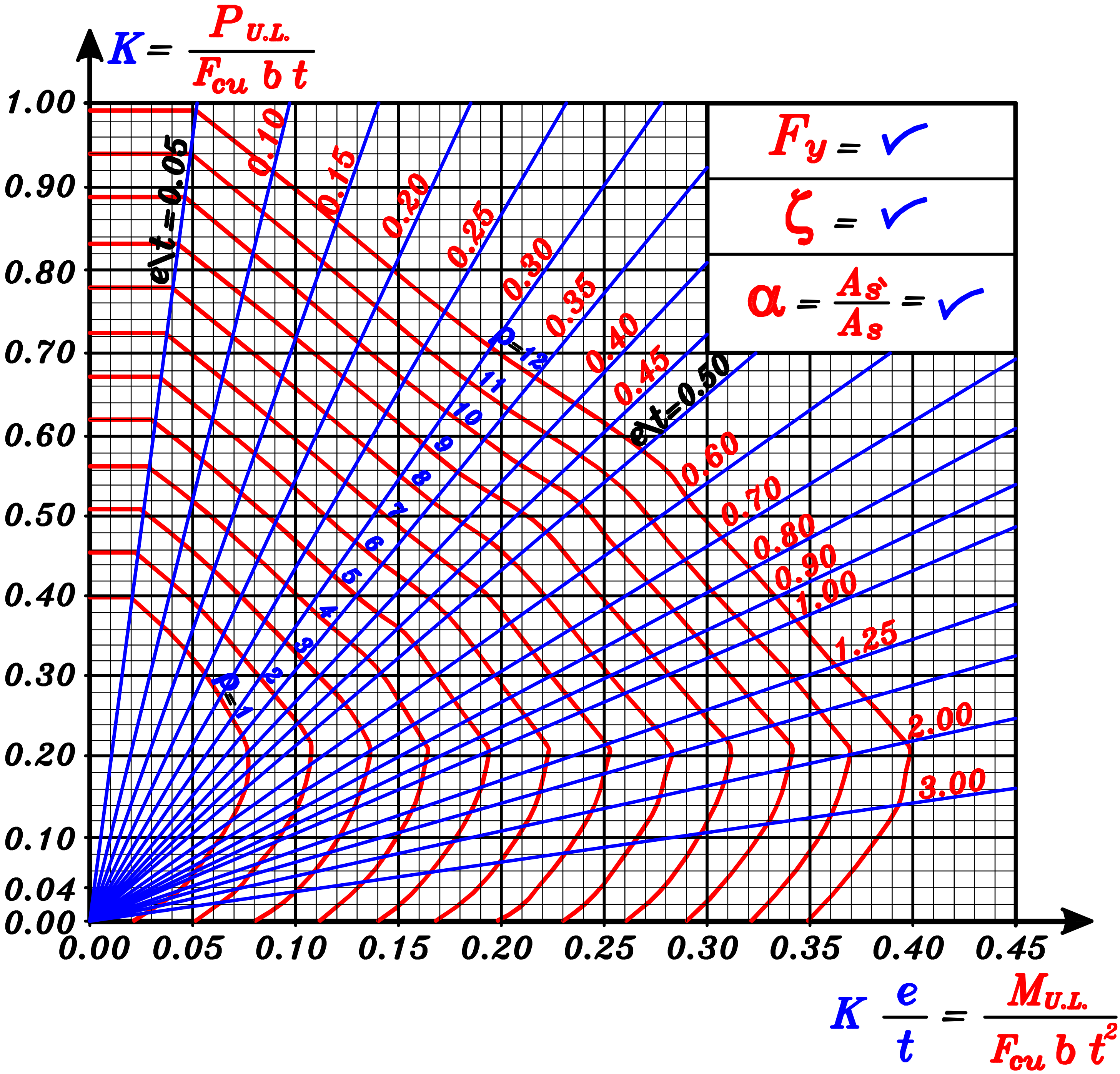
  نسبة تحدد قبل بدء ال Design و تؤخذ عاده تساوى ١

- $\zeta = \frac{t - 2Cover}{t} = \frac{\text{المسافه بين الحديد}}{\text{التخانه الكليه}}$ و تقرب للرقم الأصغر



Example: $t = 800$ mm

$$\therefore \zeta = \frac{800-100}{800} = \frac{700}{800} = 0.875 \xrightarrow{Take} \zeta = 0.8$$



$$\mu = \rho * F_{cu} * 10^{-4}$$

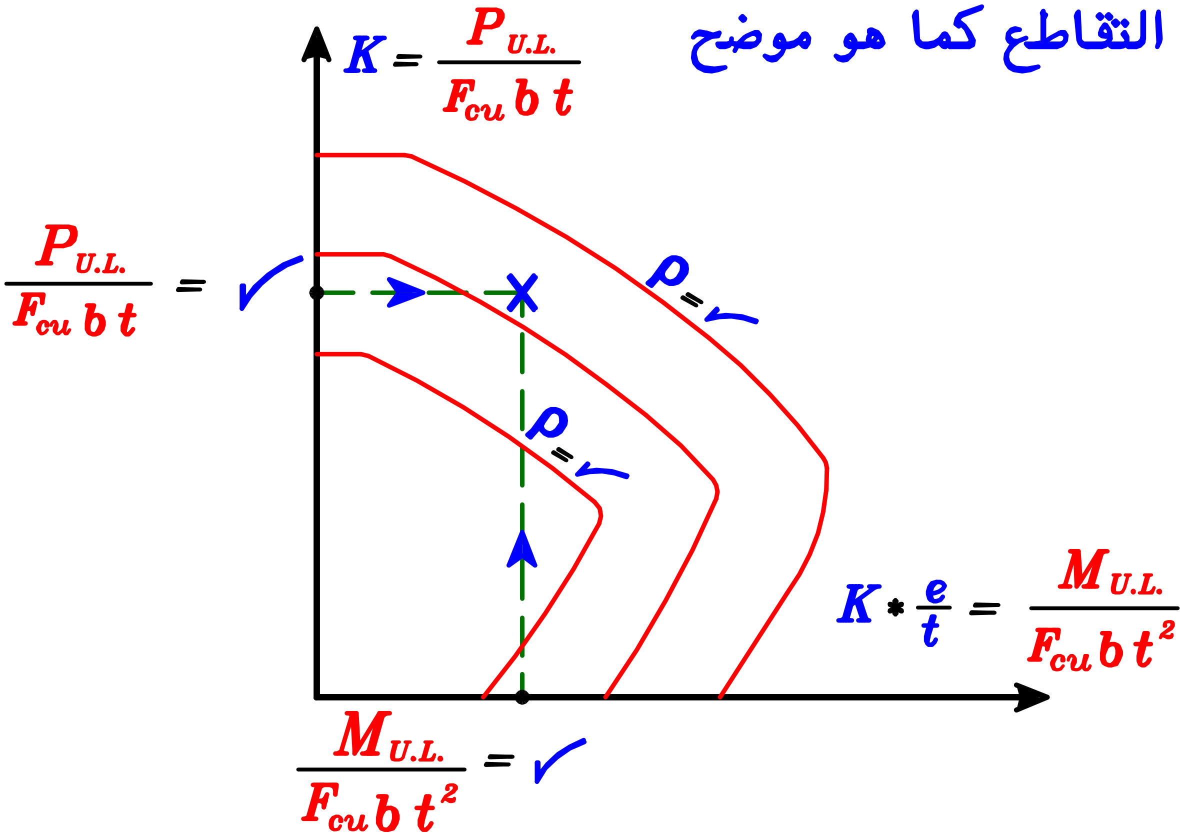
$$A_s = \mu * b * t$$

$$A_{s`} = \alpha * A_s$$

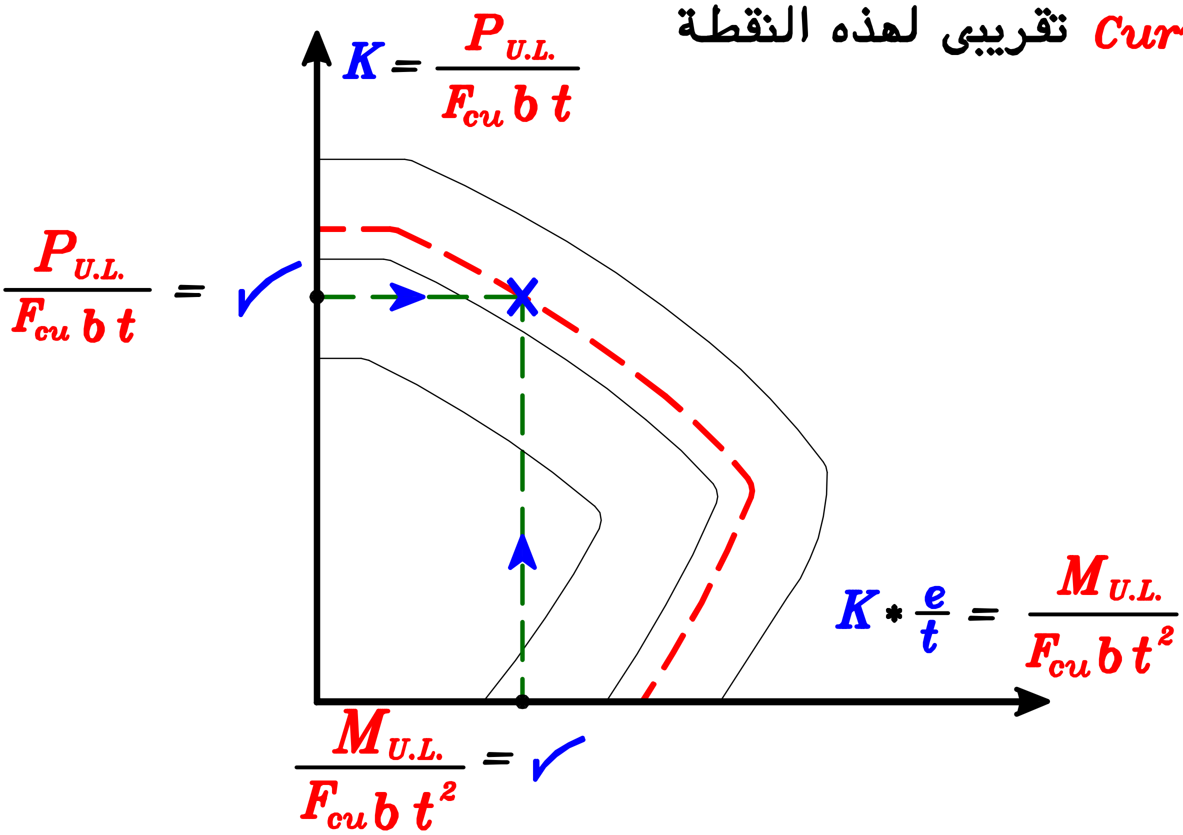
١- بعد تحديد الـ Curve بمعرفة كل من F_y , α , ζ

٢- نحدد قيمة كل من $K * \frac{e}{t} = \frac{M_{U.L.}}{F_{cu}\, b\, t^2}$, $K = \frac{P_{U.L.}}{F_{cu}\, b\, t}$

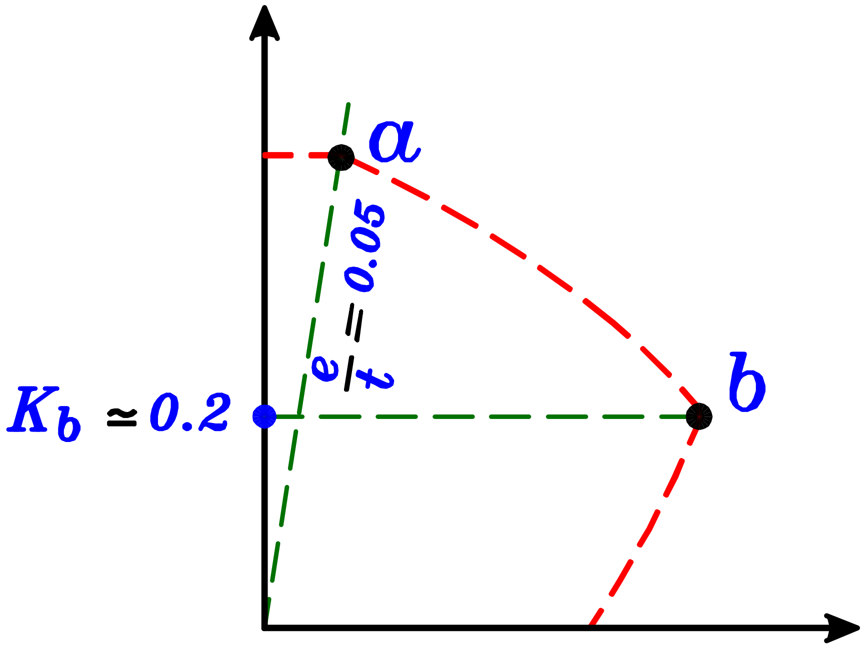
ثم نحدد نقطة التقاطع كما هو موضح



٣- ثم نرسم Curve تقريبى لهذه النقطة



٤- نحدد النقطتين a , b على هذا ال Curve
كما هو موضح بالشكل

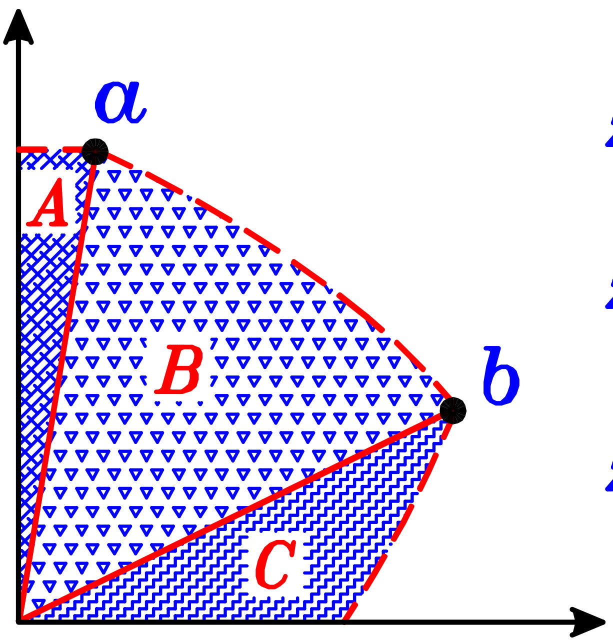


$$K_b = K_{balanced}$$

$$K_b = \frac{P_b}{F_{cu}\, b\, t} \simeq 0.2$$

حيث a هى نقطة min eccentricity
و عند هذه النقطة تكون $\frac{e}{t} = 0.05$
و نقطة b هى نقطة ال Balanced Failure

٥- من النقطتين a , b نوصل خطين الى نقطة ال origin (0,0)
و نقسم المساحة الى Zones
و نحدد طريقة ال Design

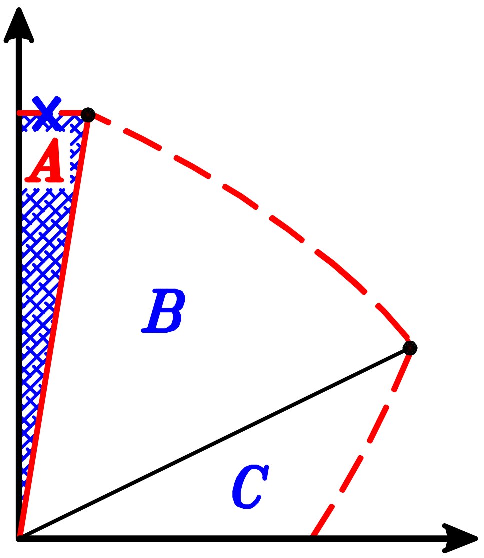


Zone A $\longrightarrow$ Design as Short Column

Zone B $\longrightarrow$ Design as Compression Failure

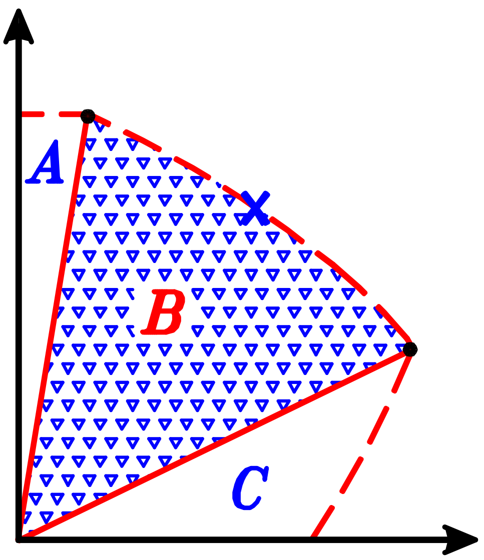
Zone C $\longrightarrow$ Design as Tension Failure

بعد تحديد نقطة تقاطع $K = \frac{P_{U.L.}}{F_{cu}\, b\, t}$, $K * \frac{e}{t} = \frac{M_{U.L.}}{F_{cu}\, b\, t^2}$



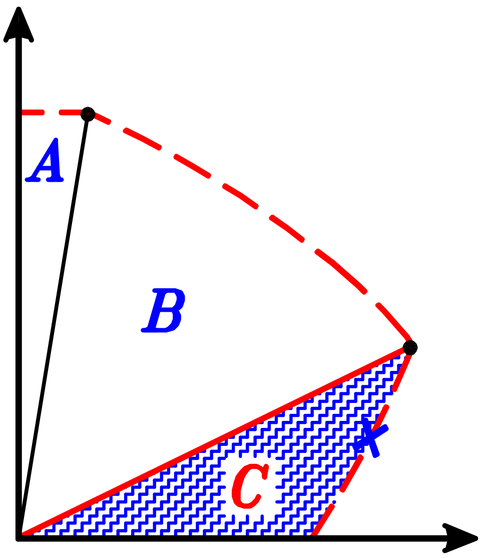
عند وجود نقطة التقاطع عند Zone A
نهمل وجود ال moment و نصمم على
ال Normal فقط

Design as Short Column using $P_{U.L.}$



عند وجود نقطة التقاطع عند Zone B
يكون أغلب القطاع علية Compression

Design as Compression Failure using Interaction Diagram



عند وجود نقطة التقاطع عند Zone C
يكون أغلب القطاع علية Tension

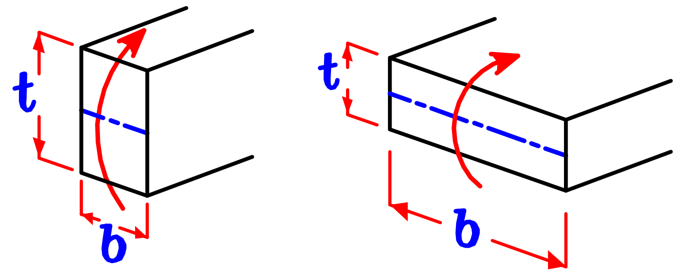
Design as Tension Failure using e_s

2- Approximate Method. ٢ - طريقة تقريبية .

(المعمول بها فى هذا الملف)

- Get $e = \frac{M_{U.L.}}{P_{U.L.}}$

t هو العرض الموازى لل moment



- Get $\frac{e}{t}$

IF $\boxed{\frac{e}{t}}$

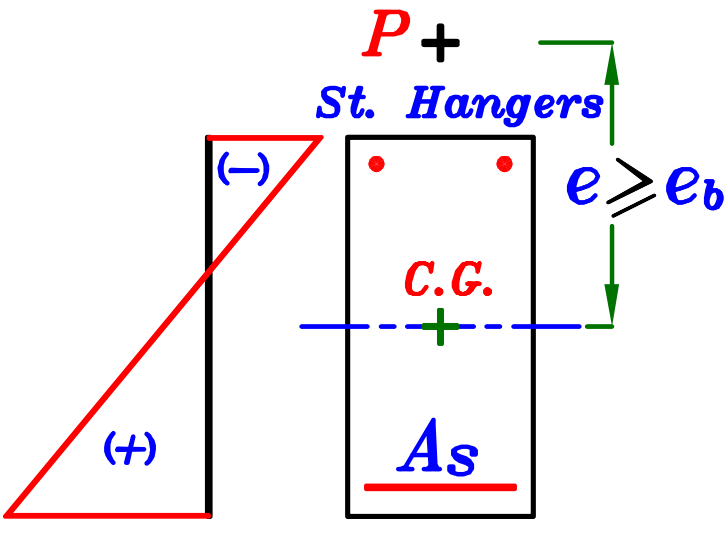
$\frac{e}{t} \geq 0.5$

Big Eccentricity

Tension Failure

معناه أن محصله القوى تؤثر خارج القطاع

$\boxed{\text{Use } e_s}$



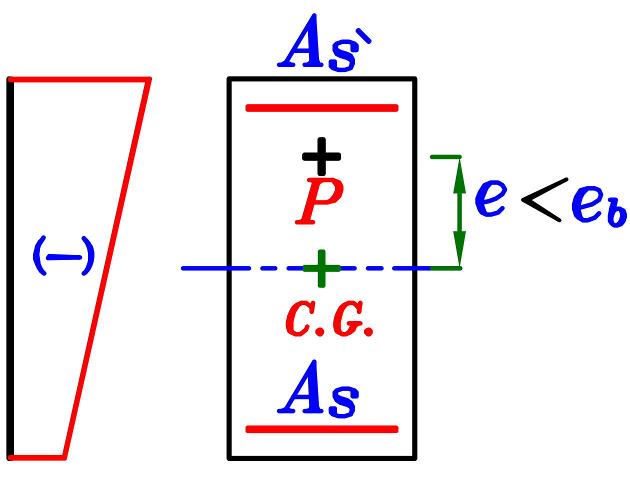
$\frac{e}{t} < 0.5$

Small Eccentricity

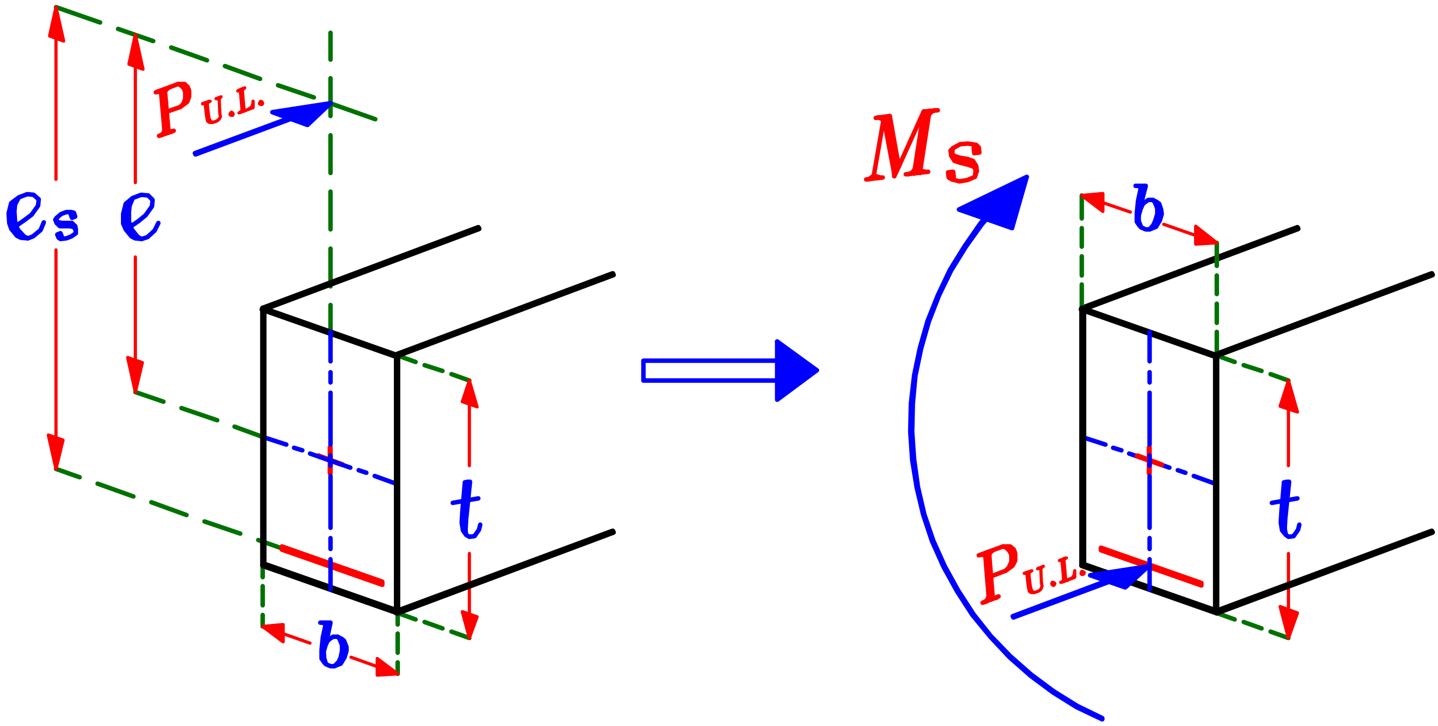
Compression Failure

معناه أن محصله القوى تؤثر داخل القطاع

$\boxed{\text{Use I.D.}}$



عندما تكون قيمه $\frac{e}{t} \geqslant 0.5$ معناه أن محصله القوى تؤثر خارج القطاع .
القطاع أقرب لقطاع الكمره منه لقطاع العمود .
أى أن جهه من الخرسانه عليها Compression و جهة عليها Tension.



Get $\boxed{e = \frac{M_{U.L.}}{P_{U.L.}}}$

Get $\boxed{e_s = e + \frac{t}{2} - c}$

حيث e هى بعد المحصله عن الـ C.G.
حيث e_s هى بعد المحصله عن الـ steel

Where: c is the Cover $\begin{cases} = 50\ mm & IF\ t \leqslant 1000\ mm \\ = 100\ mm & IF\ t > 1000\ mm \end{cases}$

- Get the moment about Tension steel

$$\boxed{M_s = P_{U.L.} * e_s}$$

- From $d = C_1 \sqrt{\dfrac{M_s}{F_{cu}\, b}}$ Get $C_1 = \checkmark \xrightarrow{get} J = \checkmark$

- Get A_s From

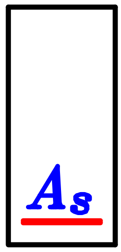
$$\boxed{A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y / \gamma_s)}}$$

- Check $A_{s_{min.}}$

Compare area of tension steel with $\left(0.225 * \dfrac{\sqrt{F_{cu}}}{F_y}\right) * b * d$

IF $A_{s_{req.}} \geq \left(0.225 * \dfrac{\sqrt{F_{cu}}}{F_y}\right) * b * d \xrightarrow{Take} A_{s_{req.}}$

IF $A_{s_{req.}} < \left(0.225 * \dfrac{\sqrt{F_{cu}}}{F_y}\right) * b * d \xrightarrow{Take} A_{s_{min.}}$



$$A_{s_{min.}} = \left\{ \begin{array}{l} \left(0.225 * \dfrac{\sqrt{F_{cu}}}{F_y}\right) * b * d \\ 1.3\, A_{s_{req.}} \end{array} \right\} \text{الأقل}$$

Stirrup Hangers.

$$\text{Stirrup Hangers} = \left. \begin{array}{l} (0.1 \rightarrow 0.2)\, A_s \\ 2\phi 12 \text{ Frames} \end{array} \right\} \text{الأكبر}$$

ملحوظه :

سواء كان ال member أفقى أو رأسى يعامل معامله الكمره

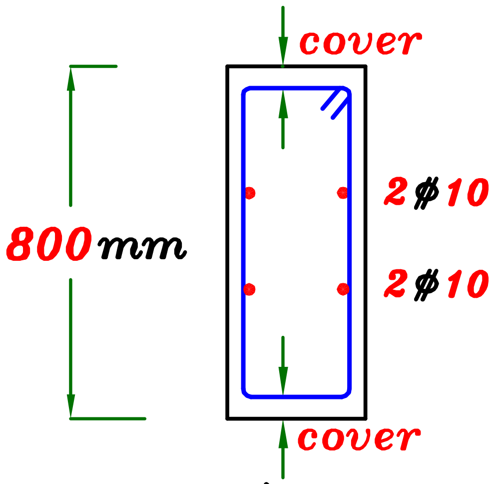
ولكن يفضل أن لا يقل ال stirrup hangers فى ال members

الرأسيه عن 0.4 A_s و هذا ليس شرط.

Shrinkage Bars. (IF the sec. in Beam.)

- توضع ال Shrinkage Bars عندما تكون $t > 700$ mm
- و قيمة ال Shrinkage Bars = $2\phi10$ at every 300 mm

Example.



IF $t = 800$ mm

$\therefore$ No. of Spacings =

$$= \frac{800-100}{300} = 2.33 = 3.0 \text{ Spacing}$$

$$= 2.0 \text{ Bars}$$

Buckling Bars. (Longitudinal Bars)

(IF the sec. in Column.)

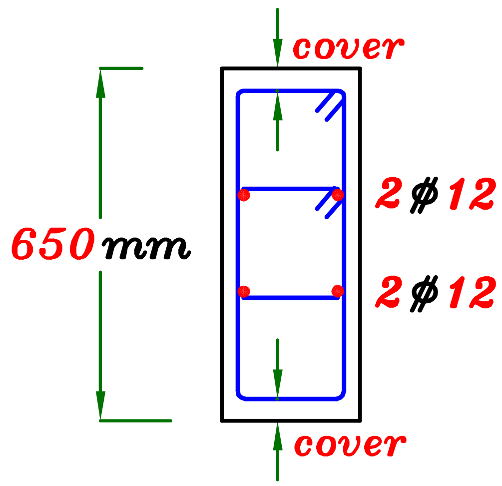
- فى الأعمده التى يؤثر عليها M & P .

يجب وضع أسياخ جانبيه تسمى Buckling Bars .

- و توضع أيضاً عندما تكون $t < 700$ mm (ليس مثل ال Shrinkage Bars)
- و قيمة ال Buckling Bars = $2\phi12$ at every 250 mm
- و توضع كانات داخليه

بحيث لا تزيد المسافه بين كل فرع كانه و الفرع الذى يليه عن ٣٠٠ مم

Example.



IF $t = 650$ mm

$\therefore$ No. of Spacings =

$$= \frac{650-100}{250} = 2.20 = 3.0 \text{ Spacing}$$

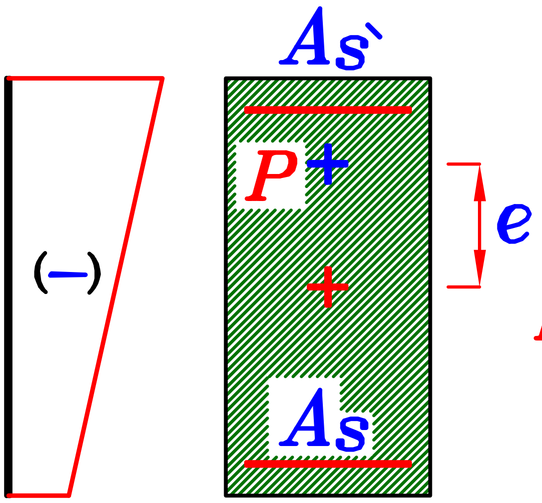
$$= 2.0 \text{ Bars}$$

Compression Failure.

عندما تكون قيمه $\frac{e}{t} < 0.5$ معناه أن محصله القوى تؤثر داخل القطاع .

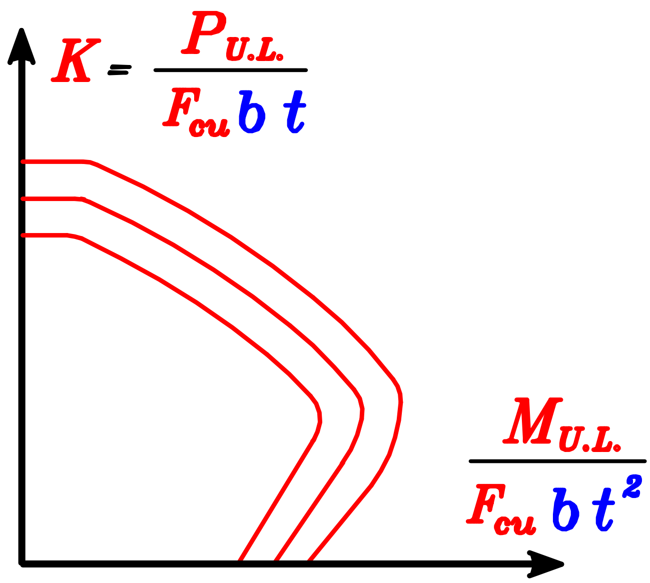
القطاع أقرب لقطاع العمود منه لقطاع الكمره .

أى يوجد *Compression* على كل القطاع .



سنحتاج لوضع تسليح فى الاتجاهين *As`* ، *As*

و نستخدم *Interaction Diagram* (*I.D.*)



و ممكن من الـ (*I.D.*) تصميم قطاعات

Big eccentricity or *small eccentricity*

أى عندما تكون $\frac{e}{t} < 0.5$ or $\frac{e}{t} > 0.5$

و لكنه فى حاله *Big eccentricity* أى $\frac{e}{t} > 0.5$ يكون غير دقيق

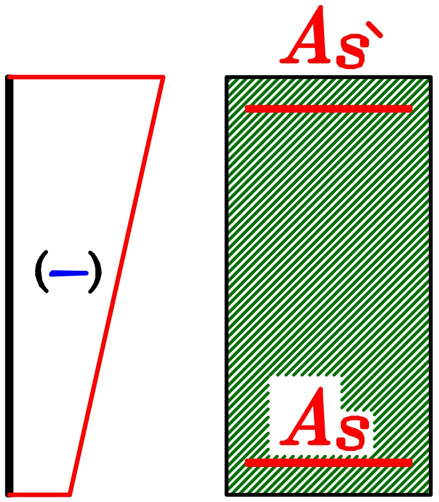
و يعطى كميات تسليح كبيره و مكلفه .

لذا يفضل استخدام الـ *Interaction Diagram* (*I.D.*) عندما

يكون القطاع *small eccentricity* أى $\frac{e}{t} < 0.5$

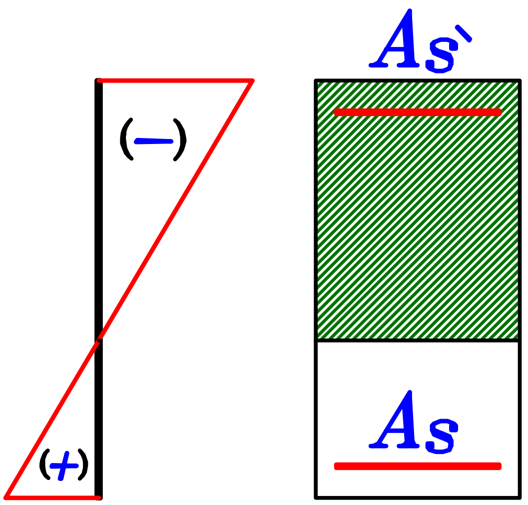
الانسب و الاوفر عند استخدام *Interaction Diagram*

ان نختار قيم α حيث $\alpha = \frac{As`}{As}$



IF $\frac{e}{t} < 0.5$

يفضل اختيار $\alpha = \frac{As`}{As} = (0.8 \rightarrow 1.0)$



IF $\frac{e}{t} > 0.5$

يفضل اختيار $\alpha = \frac{As`}{As} = (0.4 \rightarrow 1.0)$

و ان كان الافضل حساب التسليح بطريقه e_s

ملحوظه

الموجود فى كتاب *ECCS* $\boxed{\alpha = 0.8 \ \& \ 1.0}$ فقط

لذا فى هذه الملفات سنستخدم قيمه $\alpha = 0.8$ or $\alpha = 1.0$ فقط

مع العلم انه فى العمل توجد جداول $\boxed{\alpha = 0.0 \longrightarrow 1.0}$

To get A_s , $A_s`$ using Interaction Diagram.

ECCS Pages (4–20) ⟶ (4–63)

لتحديد الصفحه المطلوبه نحدد ثلاثه قيم . F_y , α , ζ

Chart Key

$F_y = \checkmark$
$\zeta = \checkmark$
$\alpha = \frac{A_s`}{A_s}$

مفتاح الجدول **Chart Key**

يوجد فى كل صفحه من صفحات الـ $I.D.$ فى الجداول مفتاح للجدول لتحديد أى جدول سوف نستخدمه

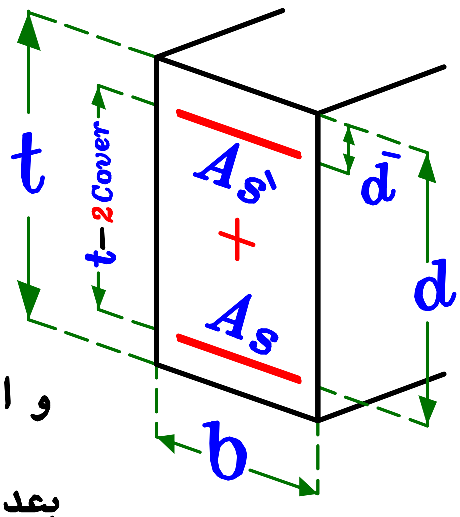
- F_y = Type of Steel
  - 240
  - 280
  - 360 ✓✓
  - 400 ✓✓

- $\alpha = \frac{A_s`}{A_s}$
  - 0.8
  - 1.0 ✓✓

و تؤخذ عاده تساوى ١

- $\zeta = \frac{d - d`}{t} = \frac{\text{المسافه بين الحديد}}{\text{التخانه الكليه}}$

$$\zeta = \frac{t - 2\,Cover}{t}$$



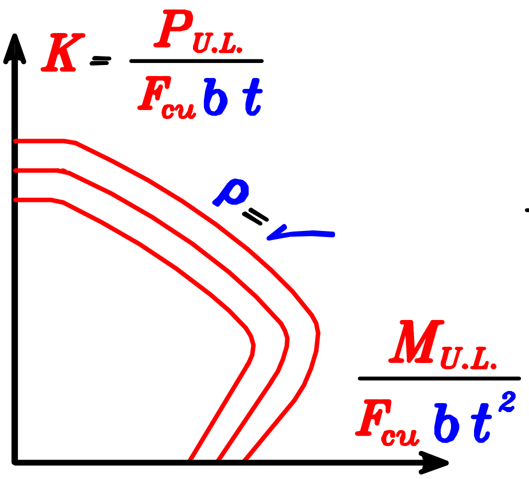
$$\boxed{\zeta = 0.7 \text{ or } 0.8 \text{ or } 0.9}$$ و الموجود فى الجداول

بعد حساب قيمة ζ اذا كانت بين رقمين تقرب للرقم الأصغر

Example. $t = 800$ mm

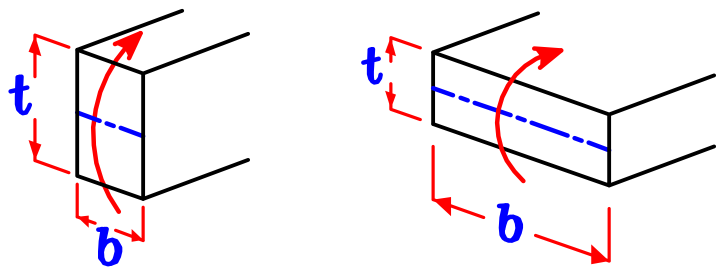
$$\therefore \zeta = \frac{800 - 100}{800} = \frac{700}{800} = 0.875 \xrightarrow{\text{Take}} \zeta = 0.8$$

بعد تحديد الـ I.D. المطلوب



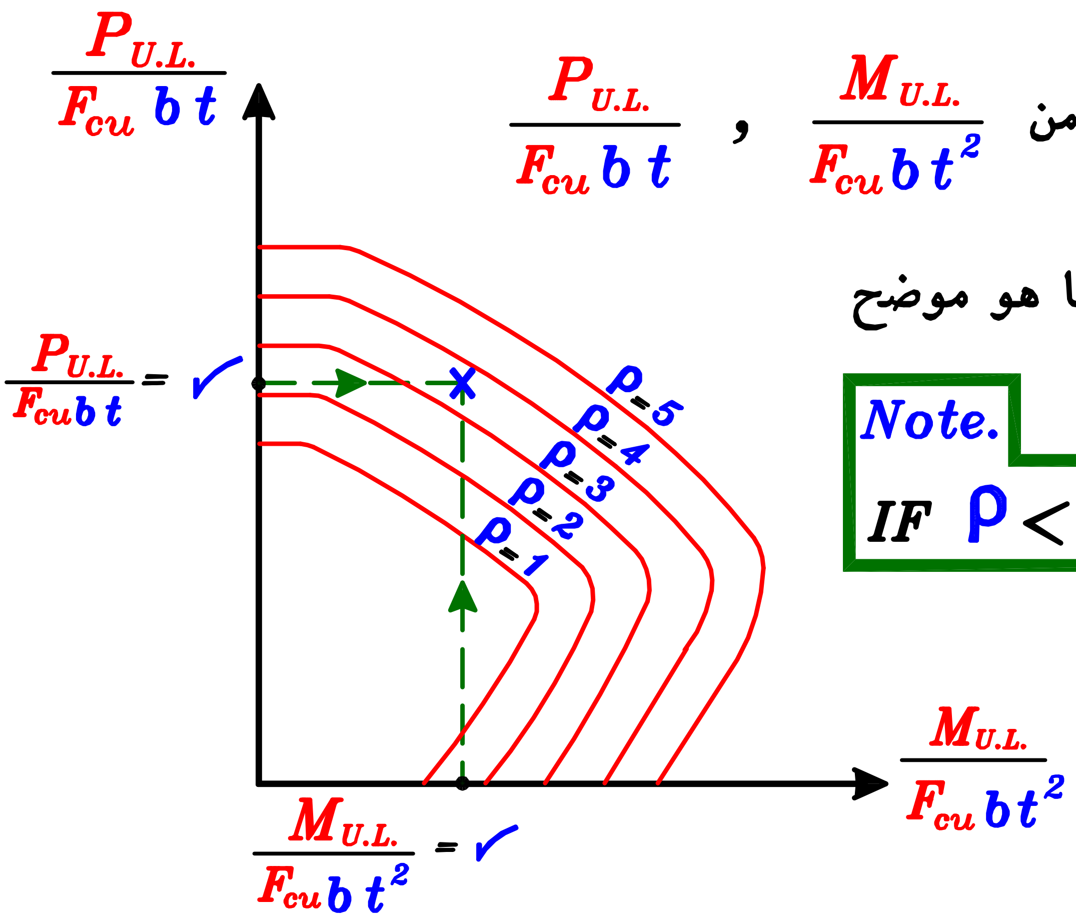
نحدد قيمه كلا من $\frac{M_{U.L.}}{F_{cu}\, b\, t^2}$, $\frac{P_{U.L.}}{F_{cu}\, b\, t}$

حيث t هو العرض المقاوم للـ moment اى العرض الموازى للـ moment



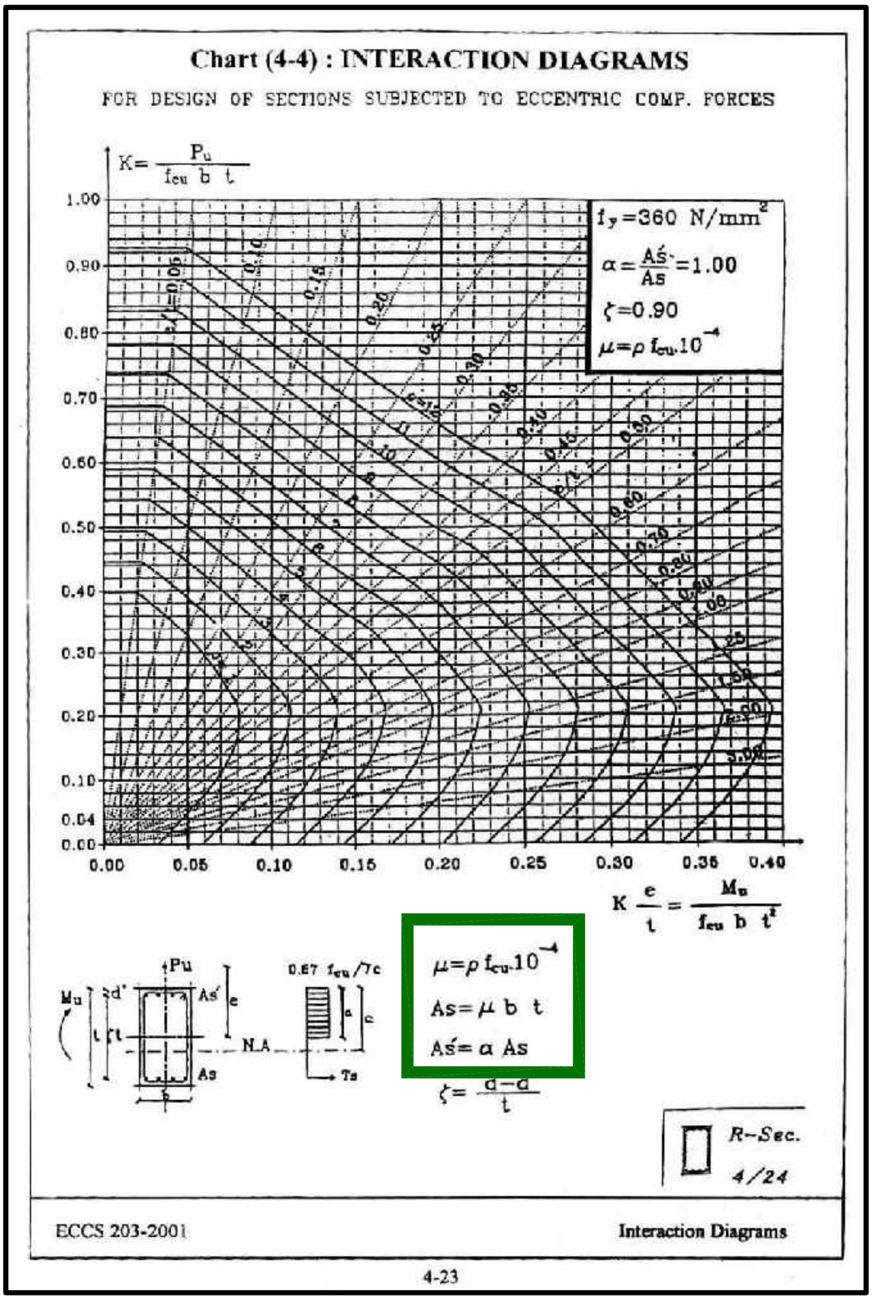
نحدد نقطه تقاطع كلا من $\frac{M_{U.L.}}{F_{cu}\, b\, t^2}$, $\frac{P_{U.L.}}{F_{cu}\, b\, t}$

ثم نحدد قيمه ρ كما هو موضح



Note.

IF $\rho < 1.0 \xrightarrow{Take} \rho = 1.0$



ثم نعوض فى المعادلات الأتيه لتحديد قيمه As , As`

$$\mu = \rho * F_{cu} * 10^{-4}$$

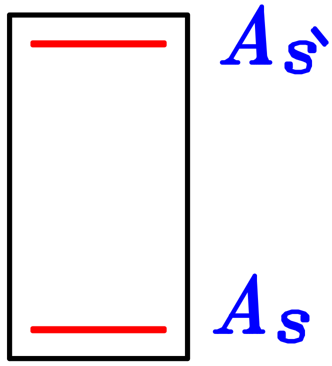
$$A_s = \mu * b * t$$

$$A_s` = \alpha * A_s$$

– Check $As_{min.}$

Calculate $As_{Total} = As + As`$

Calculate $As_{min.} = \frac{0.8}{100} * b * t$



IF $As_{Total} \geqslant As_{min.}$ $\therefore$ o.k.

IF $As_{Total} < As_{min.}$ $\xrightarrow{take}$ $As = As` = \frac{As_{min.}}{2}$

Shrinkage Bars. (IF the sec. in Beam)

– توضع ال Shrinkage Bars عندما تكون $t > 700\ mm$

– و قيمه ال Shrinkage Bars = $2 \phi 10$ at every 300 mm

Buckling Bars. (Longitudinal Bars)

(IF the sec. in Column.)

– فى الأعمده التى يؤثر عليها M & P .

يجب وضع أسياخ جانبيه تسمى Buckling Bars .

– و توضع أيضاً عندما تكون $t < 700 mm$ (ليس مثل ال Shrinkage Bars)

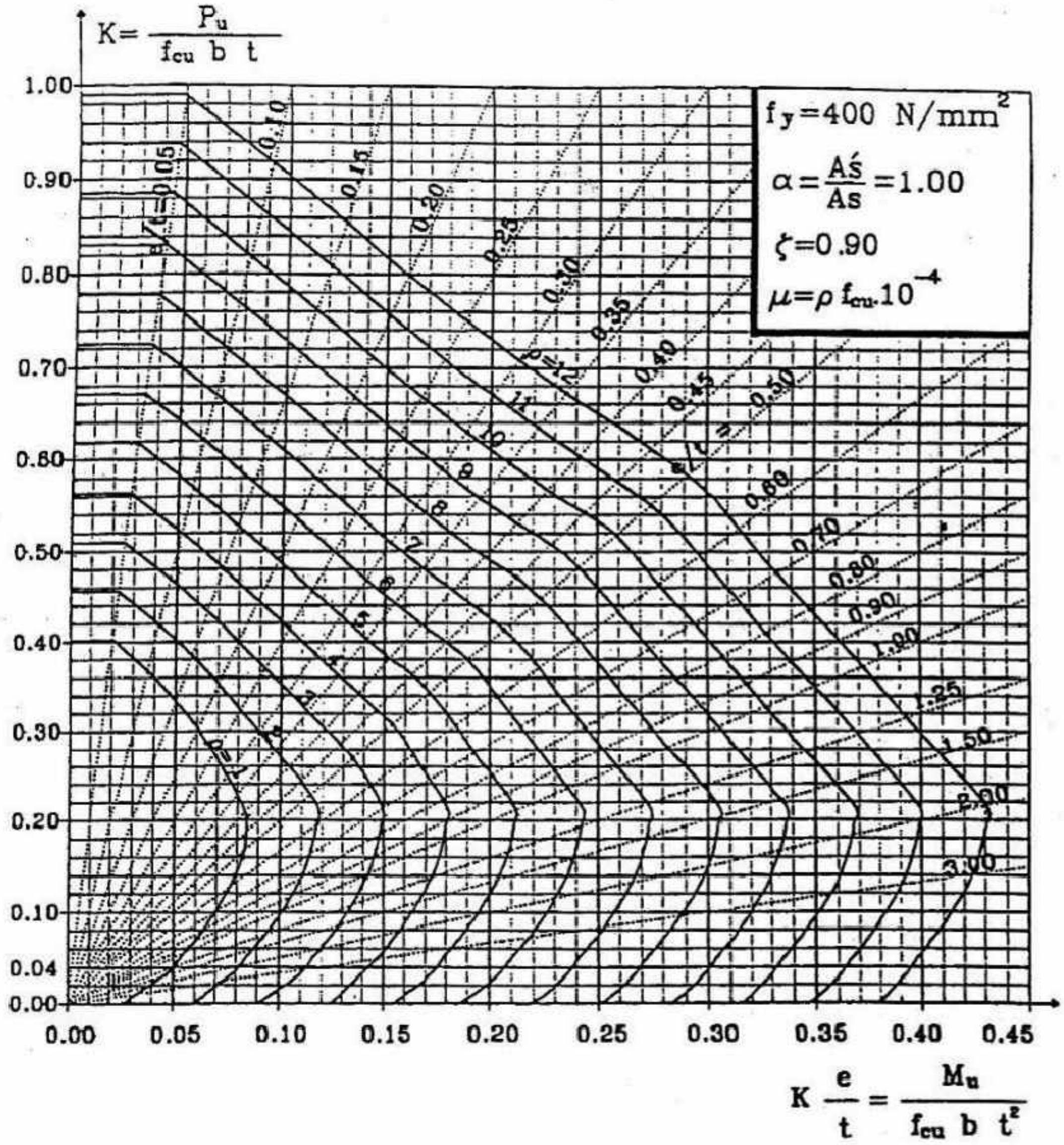
– و قيمه ال Buckling Bars = $2 \phi 12$ at every 250mm

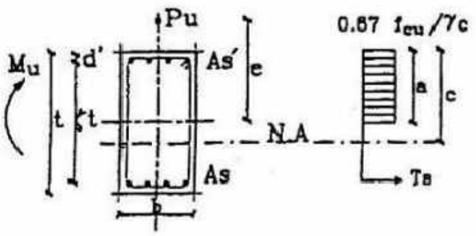
– و توضع كانات داخليه

بحيث لا تزيد المسافه بين كل فرع كانه و الفرع الذى يليه عن ٣٠٠ مم

Chart (4-1) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu = \rho\, f_{cu} \cdot 10^{-4}$

$As = \mu\, b\, t$

$A\acute{s} = \alpha\, As$

$\zeta = \dfrac{d - d'}{t}$

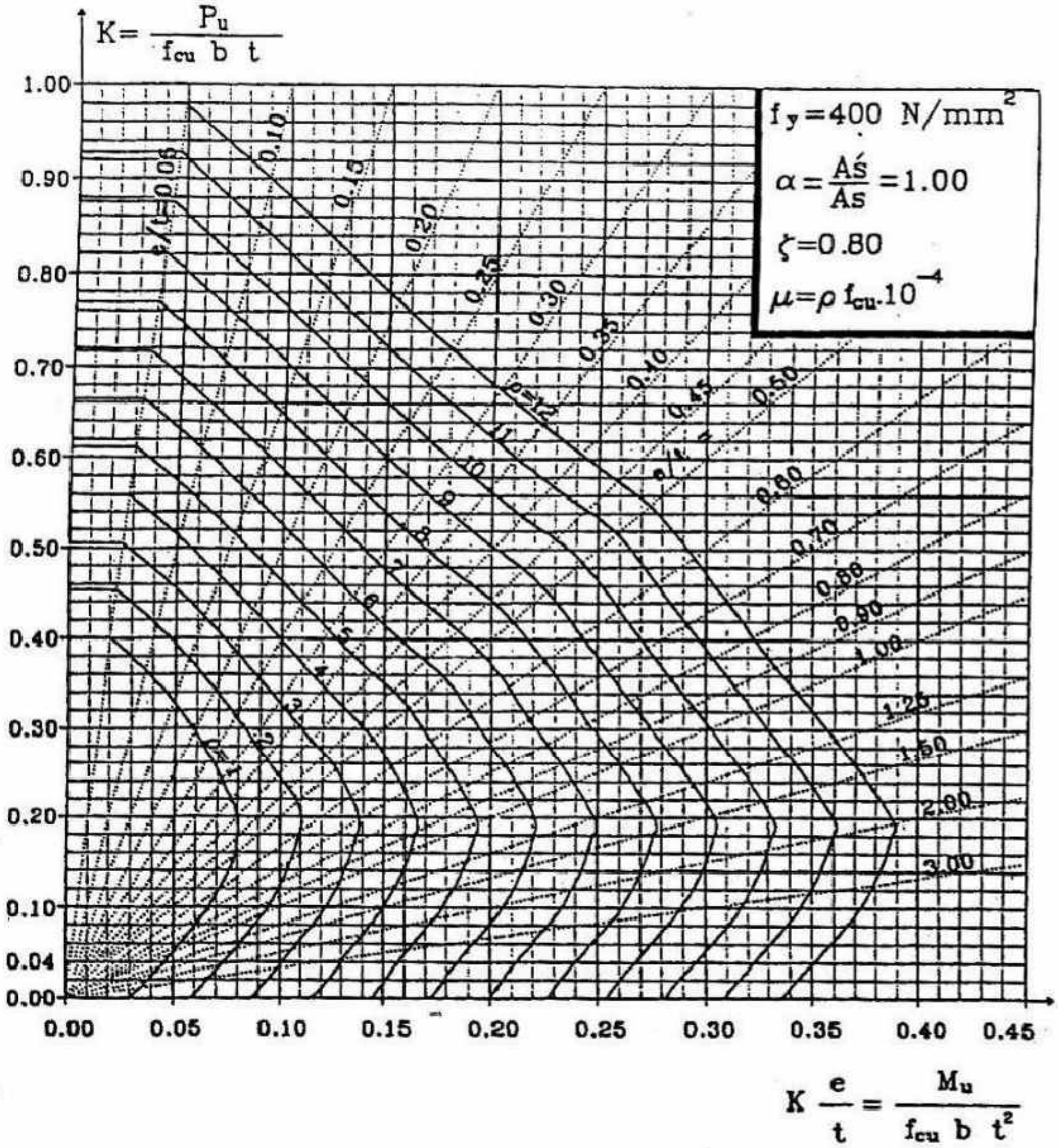
R–Sec.

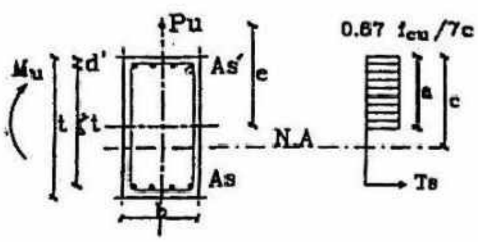
1/24

ECCS 203-2001 — Interaction Diagrams

Chart (4-2) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu=\rho\ f_{cu}.10^{-4}$

$As=\mu\ b\ t$

$As'=\alpha\ As$

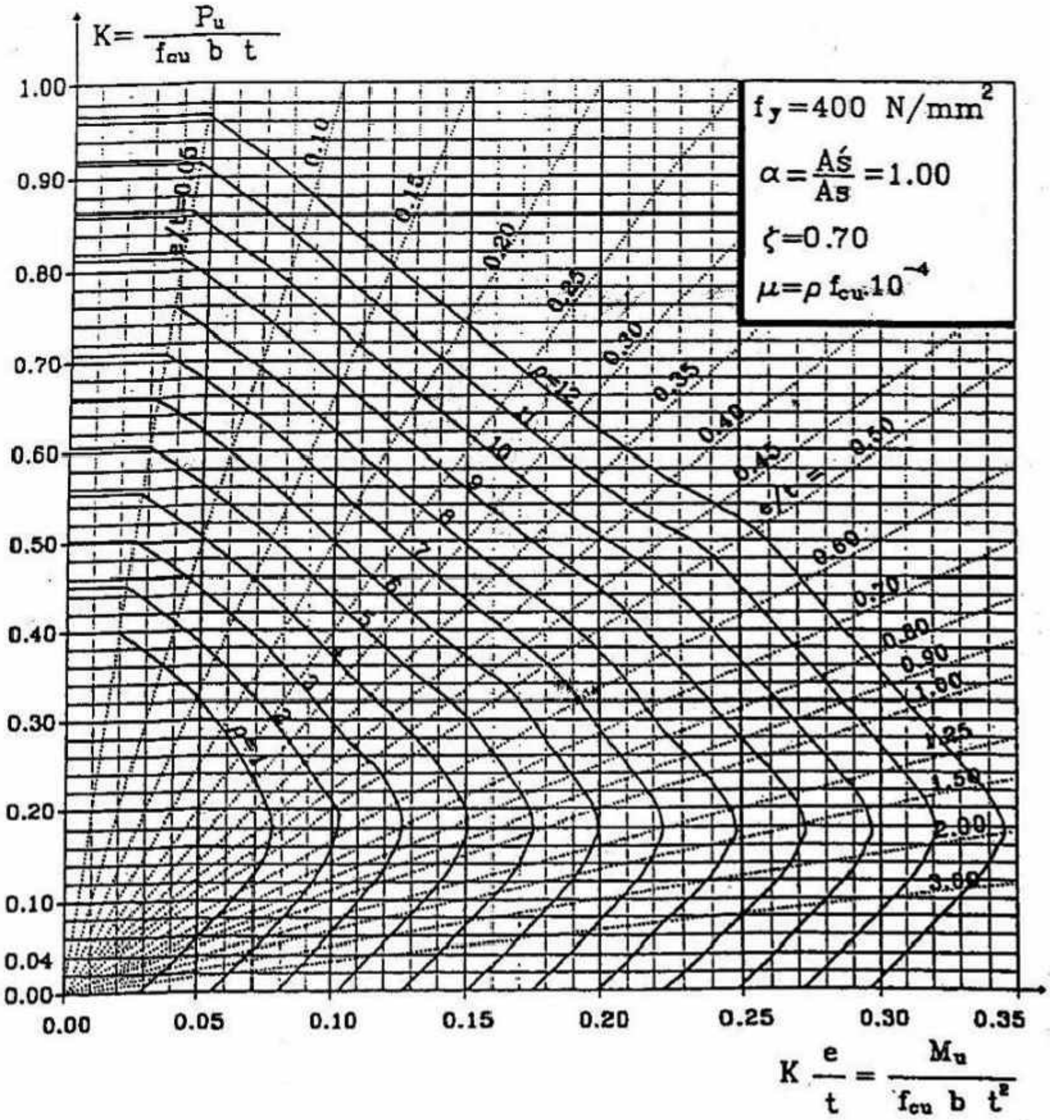
$\zeta=\frac{d-d'}{t}$

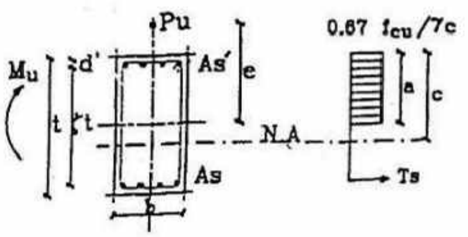
R-Sec.

2 /24

Chart (4-3) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu = \rho\ f_{cu} \cdot 10^{-4}$

$As = \mu\ b\ t$

$A\acute{s} = \alpha\ As$

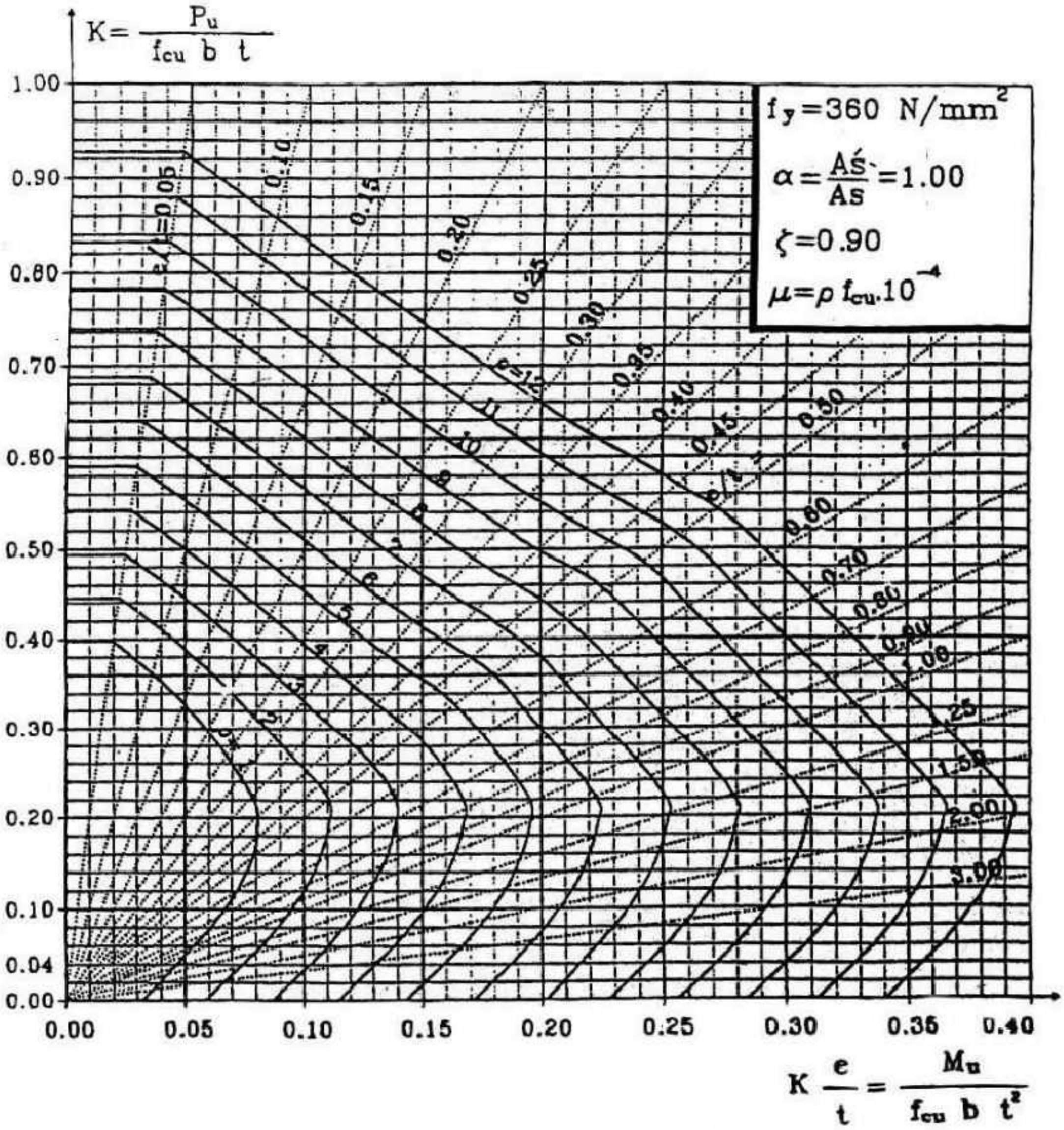
$\zeta = \frac{d - d'}{t}$

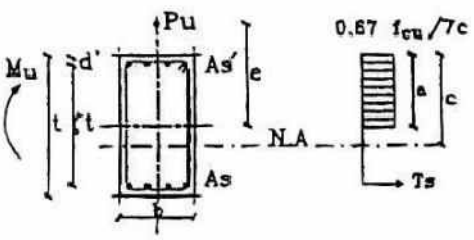
R–Sec.

3/24

Chart (4-4) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu = \rho\, f_{cu} . 10^{-4}$

$As = \mu\ b\ t$

$As' = \alpha\ As$

$\zeta = \dfrac{d - d'}{t}$

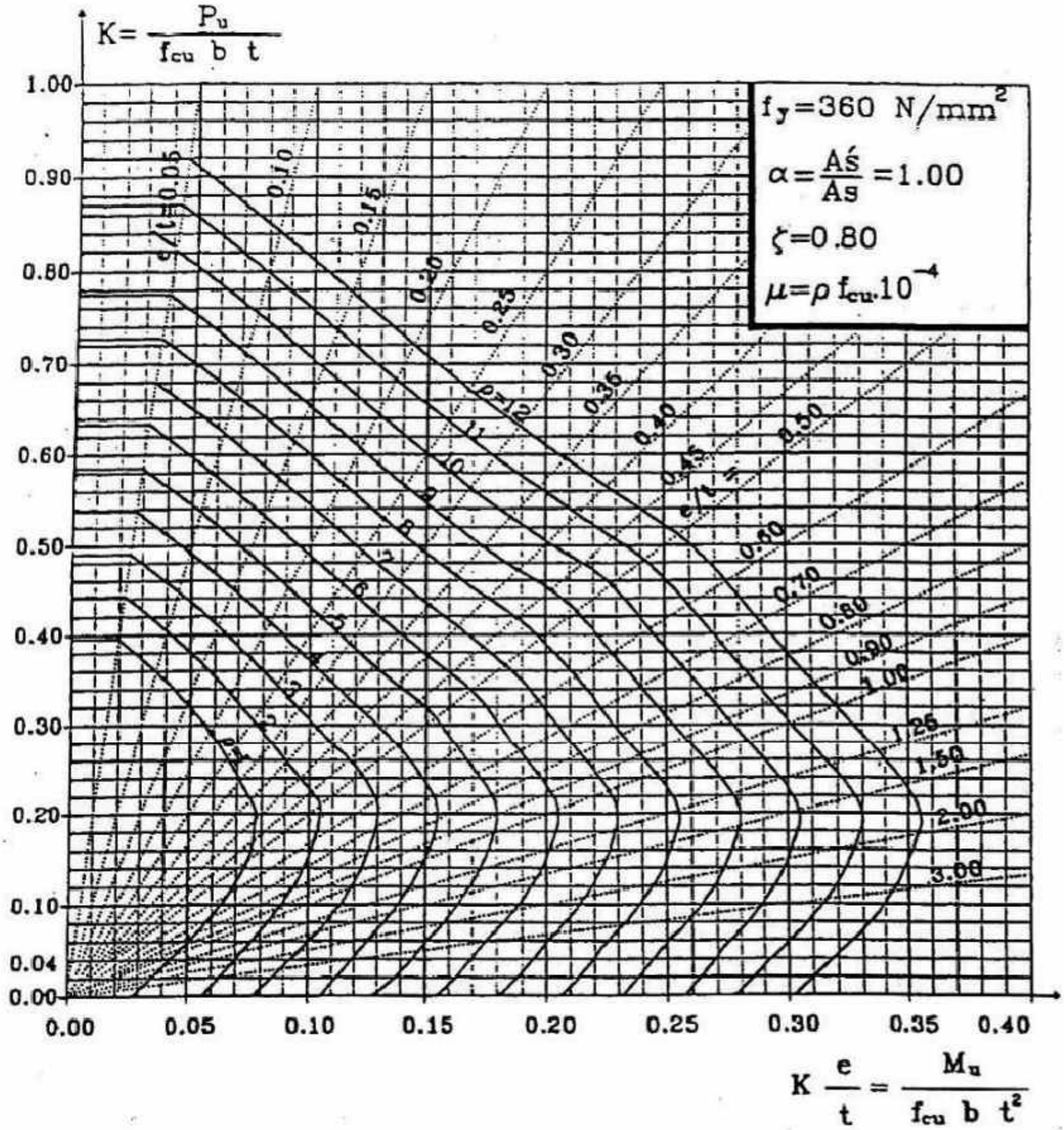
R–Sec.

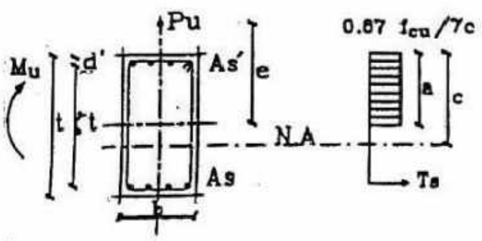
4/24

ECCS 203-2001 Interaction Diagrams

Chart (4-5) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu = \rho\, f_{cu} . 10^{-4}$

$As = \mu\ b\ t$

$As' = \alpha\ As$

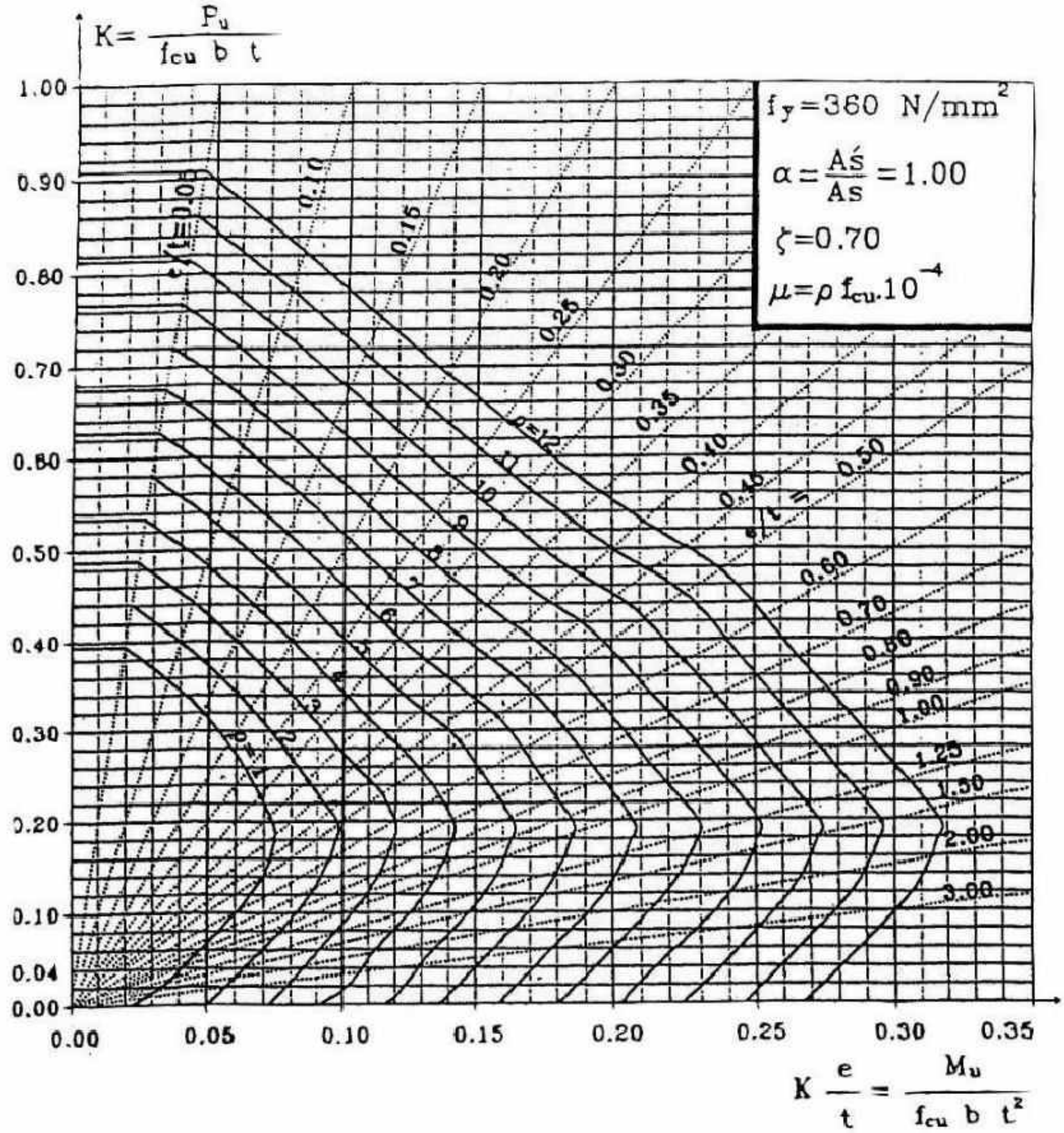
$\zeta = \frac{d - d'}{t}$

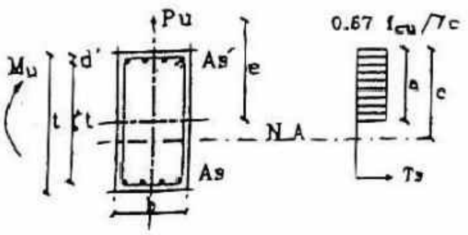
R-Sec.

5/24

Chart (4-6) : INTERACTION DIAGRAMS

FOR DESIGN OF SECTIONS SUBJECTED TO ECCENTRIC COMP. FORCES





$\mu = \rho\ f_{cu}.10^{-4}$

$As = \mu\ b\ t$

$A\acute{s} = \alpha\ As$

$\zeta = \frac{d - d'}{t}$

R-Sec.

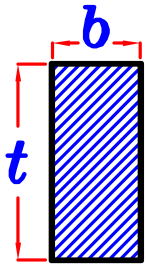
6/24

Summary of design sections subjected to M, P ملخص

① **Get Dimensions of the section.** $(b \times t)$ اذا كانت الابعاد غير موجوده

- Take $b = (300\ mm$ or $350\ mm$ or $400\ mm)$

Get $\boxed{t_1 = d_1 + cover}$ where $d_1 = 3.5\sqrt{\frac{M_{U.L.}}{F_{cu}\, b}}$



Get t_2 From $\boxed{P_{U.L.} = 0.35\,(b\,t_2)\,F_{cu} + 0.67\,\frac{(b\,t_2)}{100}\,F_y}$

- Take $t = (1.1 \rightarrow 1.3)\, t_o$ where t_o = The bigger value of t_1 & t_2

② **Check IF «P» neglected or not.**

Calculate $K = \frac{P_{U.L.}}{F_{cu}\, b\, t}$ $\xrightarrow{IF}$ IF $K \leqslant 0.04$ neglect P

$$\boxed{d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\,(b \text{ or } B)}}} \qquad \boxed{A_s = \frac{M_{U.L.}}{J\, F_y\, d}}$$

IF $K > 0.04$ don't neglect P

Design the Sec. on both M, P

- Take the same b, t From step ①

③ **Get Reinforcement A_s, A_s'**

- Get $e = \frac{M_{U.L.}}{P_{U.L.}}$ - Get $\boxed{\frac{e}{t}}$ $\boxed{t \text{ هو العرض الموازى لل moment}}$

IF $\frac{e}{t} \geqslant 0.5$

Big Eccentricity $\xrightarrow{use}$ e_s

- $e_s = e + \frac{t}{2} - c$
- $M_s = P_{U.L.} * e_s$
- From $d = C_1\sqrt{\frac{M_s}{F_{cu}\, b}}$ $\xrightarrow{Get}$ C_1 & J
- $A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y/\gamma_s)}$

Check $A_{s_{min}} = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d$

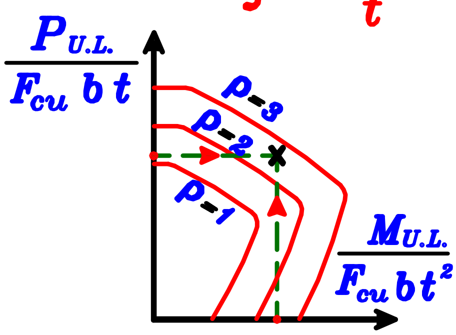
IF $\frac{e}{t} < 0.5$

small Eccentricity $\xrightarrow{use}$ I.D.

نحدد ال I.D. المناسب من كتاب الجداول على حسب كل من

F_y , $\zeta = \frac{t - 2\,Cover}{t}$, $\alpha = \frac{A_s'}{A_s} = 1$

نحدد قيمه ρ



$$\boxed{\begin{aligned} \mu &= \rho * F_{cu} * 10^{-4} \\ A_s &= \mu * b * t \\ A_s' &= \alpha * A_s \end{aligned}}$$

Check $A_{s\,Total} = A_s + A_s'$

with $A_{s_{min}} = \frac{0.8}{100} * b * t$

Design of Sections Subjected to
Bending Moment & Axial Tension Force (M, T)

Steps of Design :

1- Get Dimensions of the section. $(b \times t)$

2- Get Reinforcement A_{s1}, A_{s2}

Solution:

1- Get Dimensions of the section. $(b \times t)$

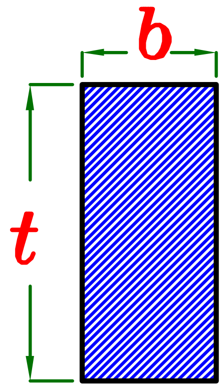
Take $b = (250\ mm \text{ or } 300\ mm \text{ or } 350\ mm \text{ or } 400\ mm)$

Get $d_o = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}}$ take $C_1 = 3.5$, $J = 0.78$

Then take $d = (0.9 \rightarrow 1.0)\, d_o$

$t = d + 50\ mm$ IF $t \leqslant 1000\ mm$

$t = d + 100\ mm$ IF $t > 1000\ mm$



2- Get Reinforcement A_{s1}, A_{s2}

Then get $e = \dfrac{M_{U.L.}}{T_{U.L.}}$ Then get $\dfrac{e}{t}$

- IF $\dfrac{e}{t} \leqslant 0.05 \longrightarrow$ neglect $M_{U.L.}$

and Design the Sec. on Tension Force only as Tie.

IF $\boxed{\dfrac{e}{t}}$

$\dfrac{e}{t} \geqslant 0.5$

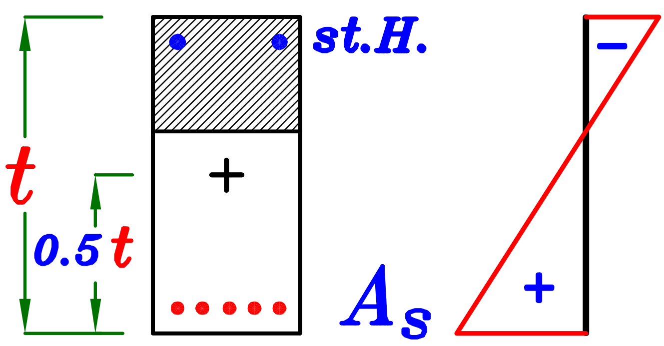
Big Eccentricity

$e \geqslant 0.5\ t$

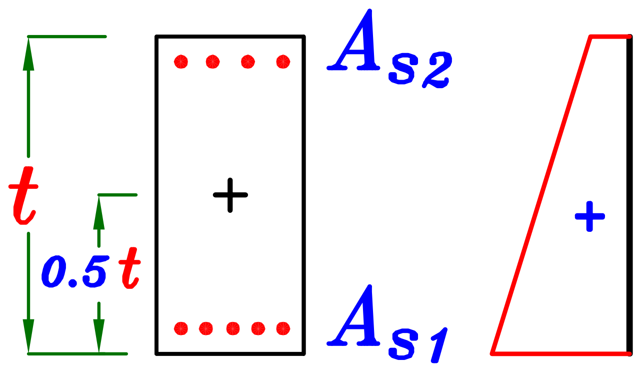
$\dfrac{e}{t} < 0.5$

Small Eccentricity

$e < 0.5\ t$



القطاع أقرب لقطاع الكمره منه لقطاع الـ Tie .



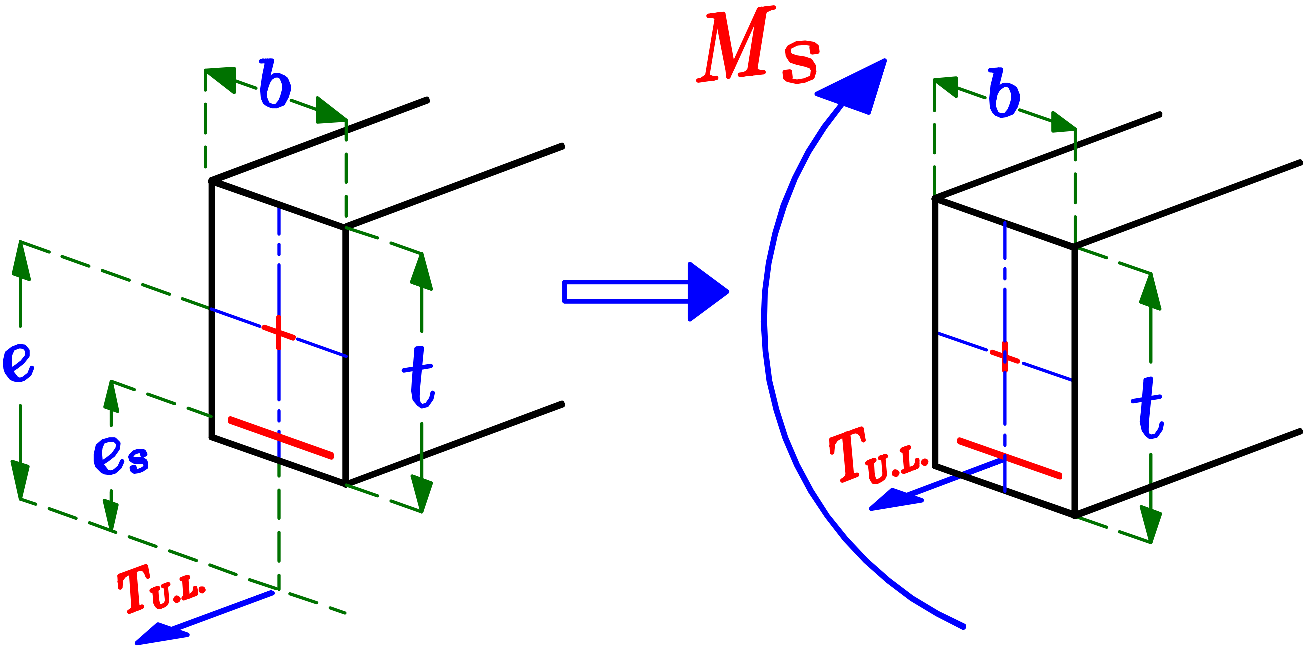
القطاع أقرب لقطاع الـ Tie منه الى قطاع الكمره .

① $\frac{e}{t} \geqslant 0.5$ Big Eccentricity.

∴ محصله القوى العموديه تكون خارج القطاع.

القطاع أقرب لقطاع الكمره منه لقطاع الـ Tie .

أى أن جهه من الخرسانه عليها Compression و جهه عليها Tension.



Get $$\boxed{e_s = e - \frac{t}{2} + c}$$

Get $$\boxed{M_s = T_{U.L.} * e_s}$$

From $d = C_1 \sqrt{\frac{M_s}{F_{cu}\, b}}$ Get $C_1 = \checkmark \xrightarrow{get} J = \checkmark$

$$\boxed{A_s = \frac{M_s}{J\, F_y\, d} + \frac{T_{U.L.}}{(F_y/\gamma_s)}}$$

Check $A_{s\,min} = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d$

Example.

$F_{cu} = 25 \ N \backslash mm^2$ st. 360/520

$M_{U.L.} = 300 \ kN.m$, $T_{U.L.} = 300 \ kN$, $b = 300 \ mm$

Req. Design the Sec.

Solution.

Take $d_o = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu} \ b}}$ $C_1 = 3.5$, $J = 0.78$

$$\therefore \ d_o = 3.5 \sqrt{\frac{300 * 10^6}{25 * 300}} = 700 \ mm$$

$$d = (0.9 \rightarrow 1.0) \ d_o = (0.9 \rightarrow 1.0) \ (700) = (630 \rightarrow 700) \ mm$$

Take $\boxed{d = 650 \ mm}$, $\boxed{t = 650 + 50 = 700 \ mm}$

$$e = \frac{M}{T} = \frac{300}{300} = 1.0 \ m$$

$$\therefore \ \frac{e}{t} = \frac{1.0}{0.7} = 1.428 > 0.5 \xrightarrow{Use} e_s$$

$$e_s = e - \frac{t}{2} + c = 1.0 - \frac{0.70}{2} + 0.05 = 0.70 \ m$$

$$M_s = T * e_s = 300 * 0.70 = 210 \ kN.m$$

$$\therefore \ d = C_1 \sqrt{\frac{M_s}{F_{cu} \ b}}$$

$$\therefore \ 650 = C_1 \sqrt{\frac{210 * 10^6}{25 * 300}} \rightarrow C_1 = 3.884 \rightarrow J = 0.798$$

$$\therefore A_s = \frac{M_s}{J\,F_y\,d} + \frac{T_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{210 * 10^6}{0.798 * 360 * 650} + \frac{300 * 10^3}{(360 \backslash 1.15)} = 2082.9 \ mm^2$$

– Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2082.9 \ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 300 * 650 = 609.3 \ mm^2$$

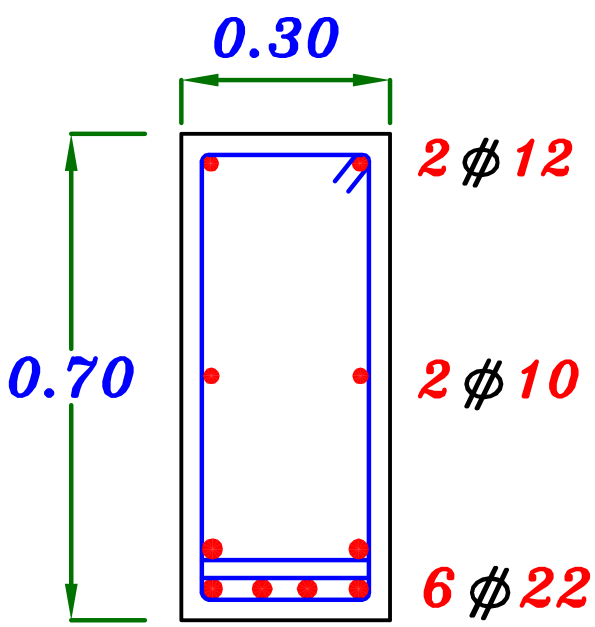
$$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$$

$\therefore$ Take $A_s = A_{s_{req.}} = 2082.9 \ mm^2$ $\quad 6 \phi 22$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{300 - 25}{22 + 25} = 5.85 = 5.0$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 2082.9$

$2 \phi 12$



② $\frac{e}{t} < 0.5$ Small Eccentricity.

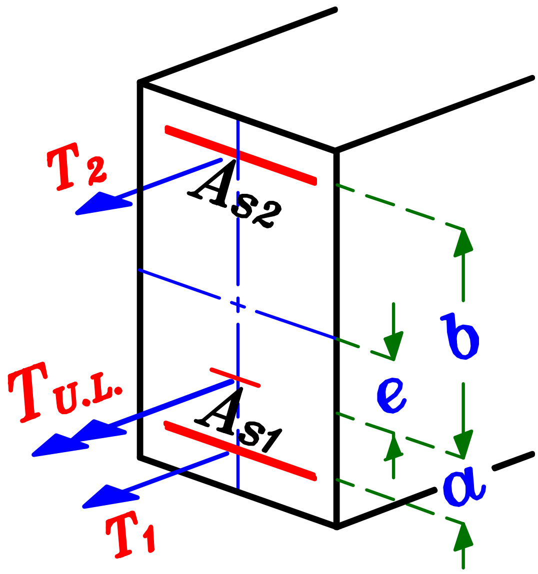
∴ محصله القوى العموديه تكون داخل القطاع.
القطاع أقرب لقطاع ال Tie منة لقطاع الكمره .
أى أن الخرسانه عليها Tension من الجهتين .
و تكون الخرسانه مشرخه من الجهتين و يقاوم الحديد كل ال Tension .

$$\boxed{a = \frac{t}{2} - c - e}$$

a هى بعد المحصله عن الحديد الاقرب لها

$$\boxed{b = \frac{t}{2} - c + e}$$

b هى بعد المحصله عن الحديد الابعد عنها

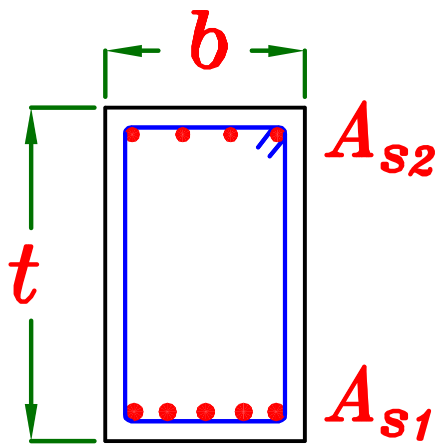


نحسب مركبتين الشد T_1 و T_2 عند الحديد القريب و البعيد عن المحصله
و منهم نحسب مساحه الحديد المطلوبه لحمل هذه القوى A_{s1} و A_{s2}

بأخذ العزم عند T_2

$$\because \; T_1\,(a+b) = T\,(b) \xrightarrow{Get} T_1$$

$$\because \; T = T_1 + T_2 \xrightarrow{Get} T_2$$



$$\boxed{A_{s1} = \frac{T_1}{(F_y/\gamma_s)}} \qquad \boxed{A_{s2} = \frac{T_2}{(F_y/\gamma_s)}}$$

دائما ال T_1 الكبيره جهه ال moment

Example.

$F_{cu} = 25\ N\backslash mm^2$ st. 360/520

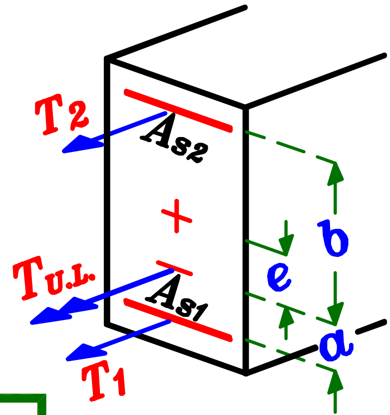
$M_{U.L.} = 100\ kN.m$, $T_{U.L.} = 600\ kN$, $b = 300\ mm$, $t = 500\ mm$

Req. Design the Sec.

Solution.

$$e = \frac{M}{T} = \frac{100}{600} = 0.1667\ m$$

$$\therefore \frac{e}{t} = \frac{0.1667}{0.50} = 0.333 < 0.5 \longrightarrow$$ Small Eccentricity.



$$a = \frac{t}{2} - c - e = \frac{0.50}{2} - 0.05 - 0.1667 = 0.033\ m$$

$$b = \frac{t}{2} - c + e = \frac{0.50}{2} - 0.05 + 0.1667 = 0.3667\ m$$

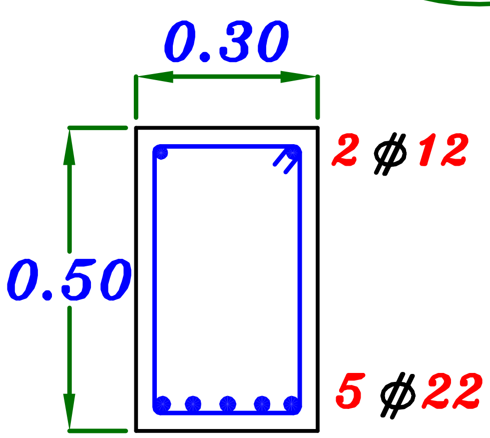
$$\because T_1\ (a+b) = T\ (b)$$ بأخذ العزم عند T_2

$$T_1\ (0.40) = 600\ (0.366) \longrightarrow \boxed{T_1 = 550\ kN}$$

$$\because T = T_1 + T_2 \quad \therefore 600 = 550 + T_2 \longrightarrow \boxed{T_2 = 50\ kN}$$

$$A_{S1} = \frac{T_1}{(F_y/\gamma_s)} = \frac{550 * 10^3}{(360\backslash 1.15)} = 1757\ mm^2$$ 5 ϕ 22

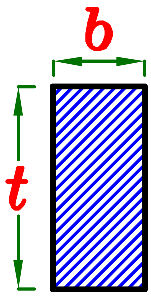
$$A_{S2} = \frac{T_1}{(F_y/\gamma_s)} = \frac{50 * 10^3}{(360\backslash 1.15)} = 159.7\ mm^2$$ 2 ϕ 12



Summary of design sections subjected to M, T ملخص

① Get Dimensions of the section. $(b \times t)$ اذا كانت الابعاد غير معطاه

- Take $b = (250\ mm$ or $300\ mm$ or $350\ mm$ or $400\ mm)$



Get $\boxed{t = d_o + cover}$ where $d_o = 3.5\sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}}$

② Get Reinforcement A_{s1}, A_{s2}

- Get $e = \dfrac{M_{U.L.}}{T_{U.L.}}$ - Get $\boxed{\dfrac{e}{t}}$

$\boxed{t \text{ هو العرض الموازى لل moment}}$

IF $\dfrac{e}{t} \geq 0.5$

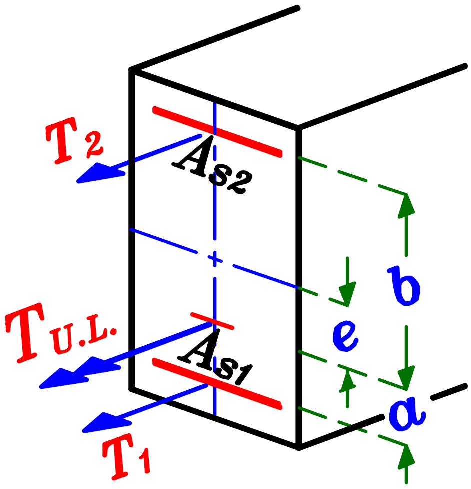
Big Eccentricity $\xrightarrow{use}$ e_s

- $e_s = e - \dfrac{t}{2} + c$
- $M_s = T_{U.L.} * e_s$
- From $d = C_1\sqrt{\dfrac{M_s}{F_{cu}\, b}} \xrightarrow{Get} C_1 \,\&\, J$
- $A_s = \dfrac{M_s}{J F_y d} + \dfrac{T_{U.L.}}{(F_y/\gamma_s)}$

Check $A_{s\,min} = \left(0.225 * \dfrac{\sqrt{F_{cu}}}{F_y}\right) b\, d$

IF $\dfrac{e}{t} < 0.5$

small Eccentricity



$\boxed{a = \dfrac{t}{2} - c - e}$ $\boxed{b = \dfrac{t}{2} - c + e}$

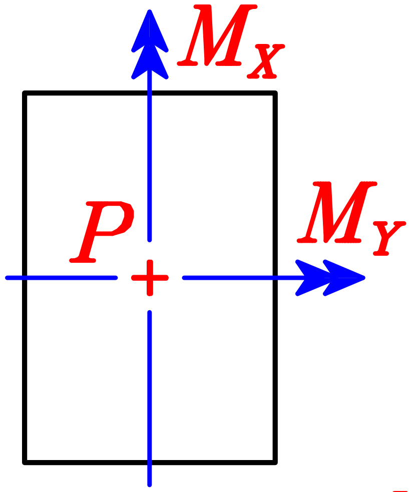
$T_1\,(a+b) = T\,(b) \xrightarrow{Get} T_1$

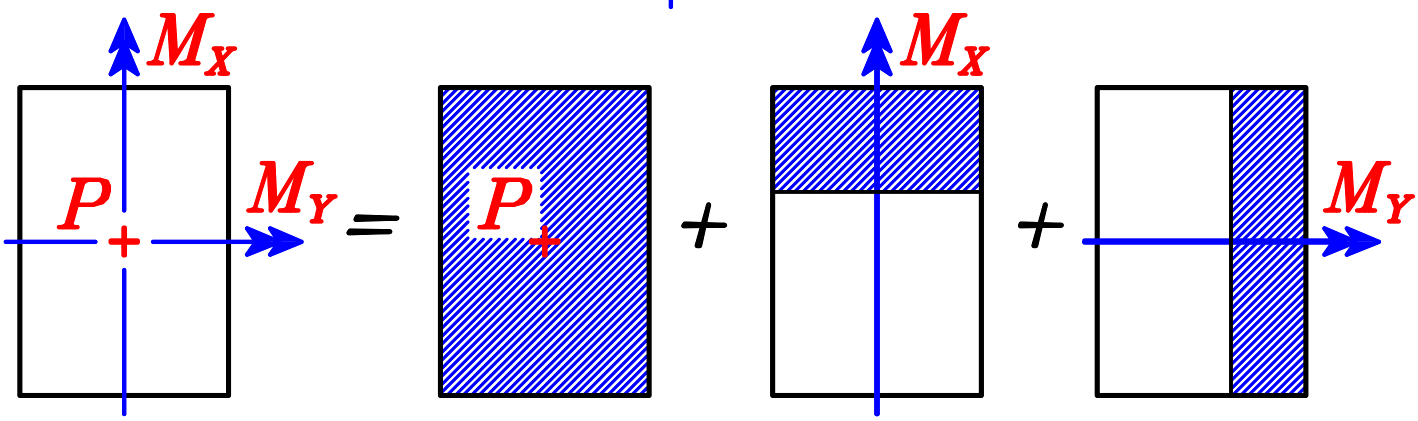
$T = T_1 + T_2 \xrightarrow{Get} T_2$

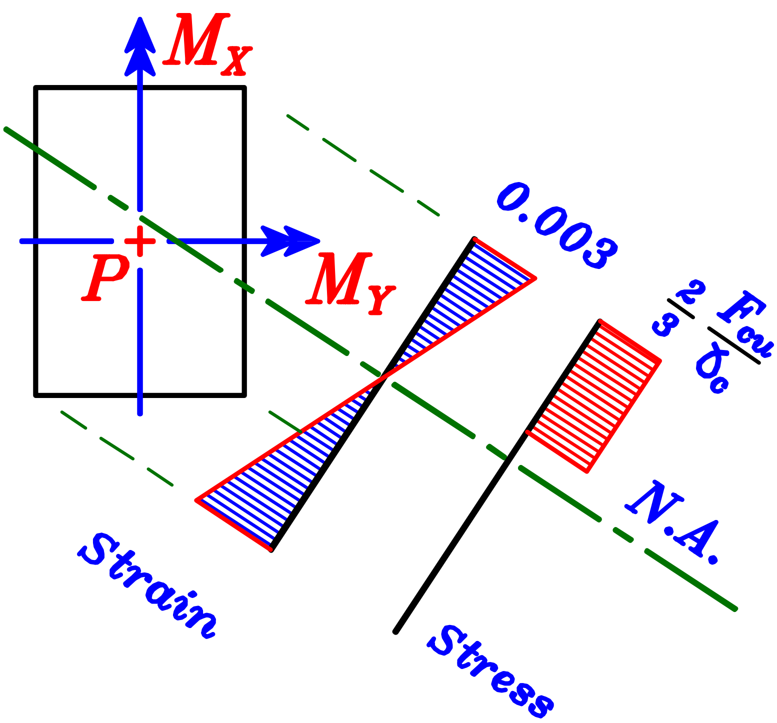
$\boxed{A_{s1} = \dfrac{T_1}{(F_y/\gamma_s)}}$ $\boxed{A_{s2} = \dfrac{T_2}{(F_y/\gamma_s)}}$

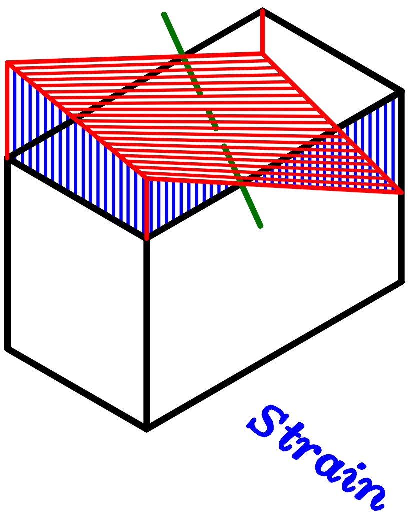
Introduction.

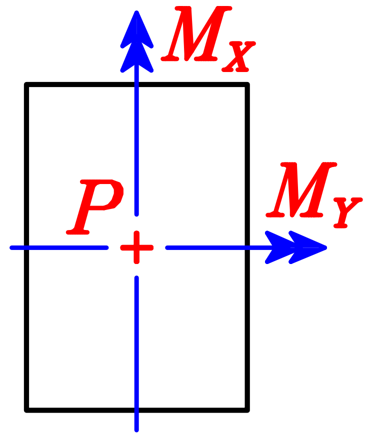
هو قطاع معرض لقوى ضغط و عزم فى الاتجاهين Bi-Axial Moment











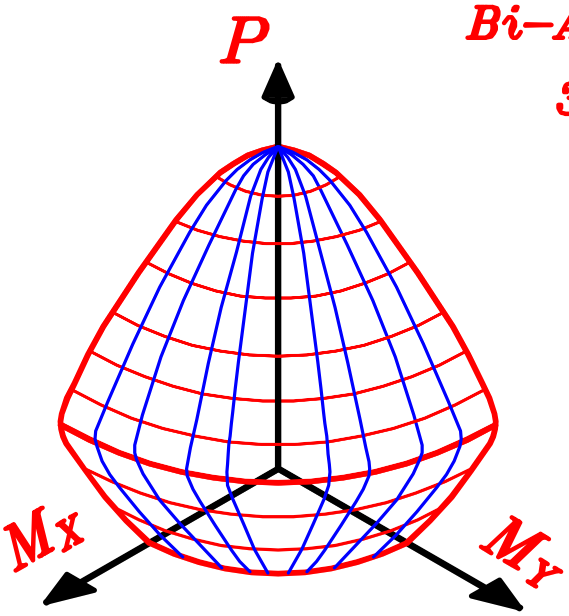
لتصميم قطاع **Bi-Axial Moment**

توجد عده طرق :

منها التصميم بـ **First Principles**

و هى صعبه جدا و لن يتم دراستها فى هذا الملف .

و سندرس فقط التصميم بال **Interaction Diagram (I.D.)**

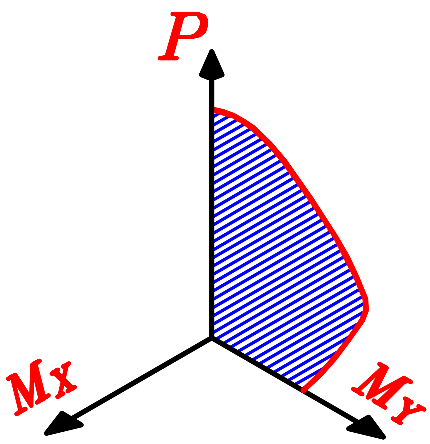


3 Dimensional (I.D.)

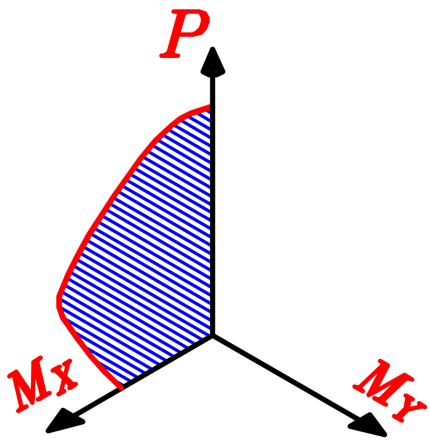
ال **(I.D.)** للقطاعات ال **Bi-Axial Moment**
المفروض أن يكون ثلاثى الابعاد **3-D (I.D.)**

بحيث ان كل نقطه تتكون من P, M_X, M_Y
اذا كانت موجوده داخل ال **3-D (I.D.)**
يكون القطاع **Safe**

و اذا كانت النقطه التى تتكون
من P, M_X, M_Y خارج ال **3-D (I.D.)**
يكون القطاع **Unsafe**

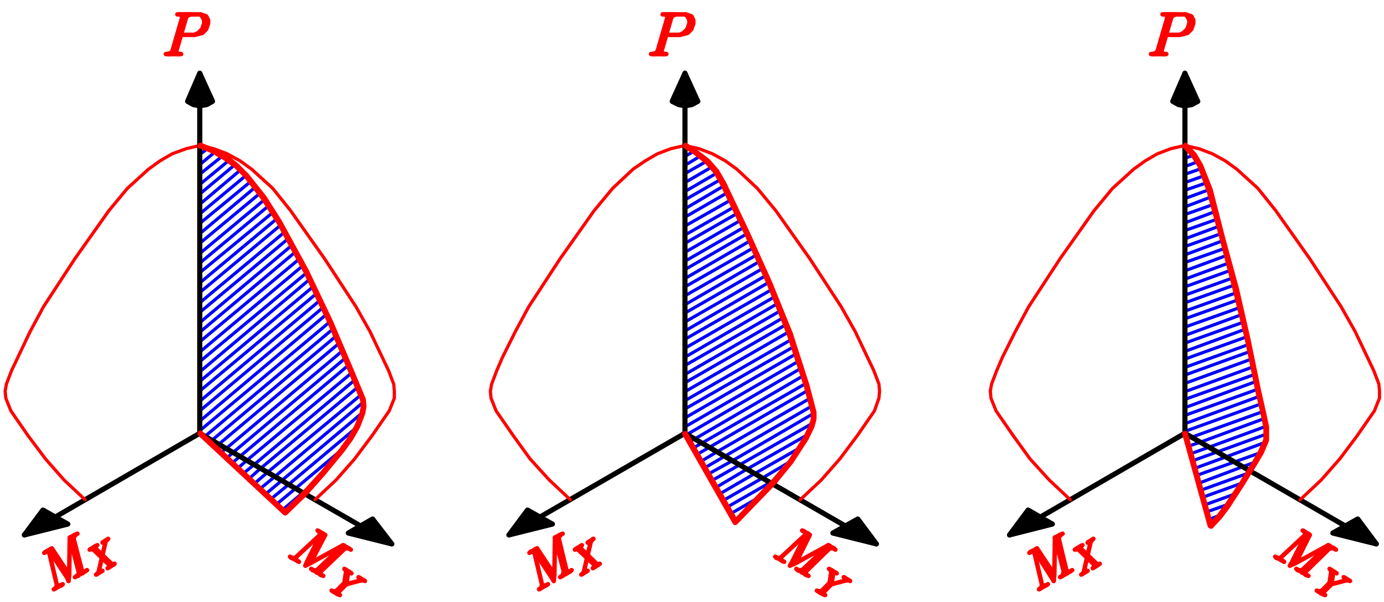


عندما يؤثر على القطاع P, M_X فقط
أى $M_Y = Zero$ سنحتاج لـ **(I.D.)**
يسمى **Uniaxial (I.D.)**



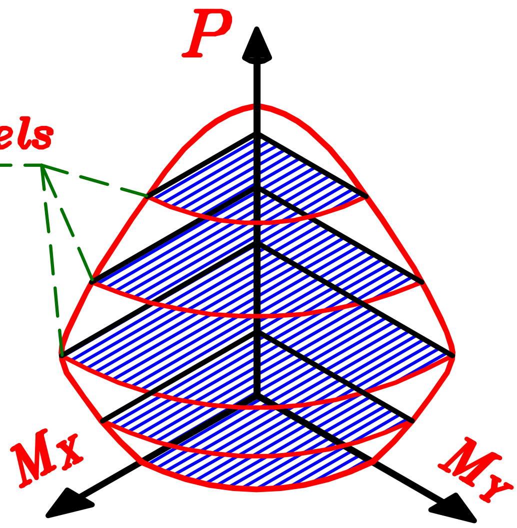
عندما يؤثر على القطاع P, M_Y فقط
أى $M_X = Zero$ سنحتاج لـ **(I.D.)**
يسمى **Uniaxial (I.D.)**

ستتغير زاويه ال *I.D.* مع تغير قيمه كلا من M_Y و M_X



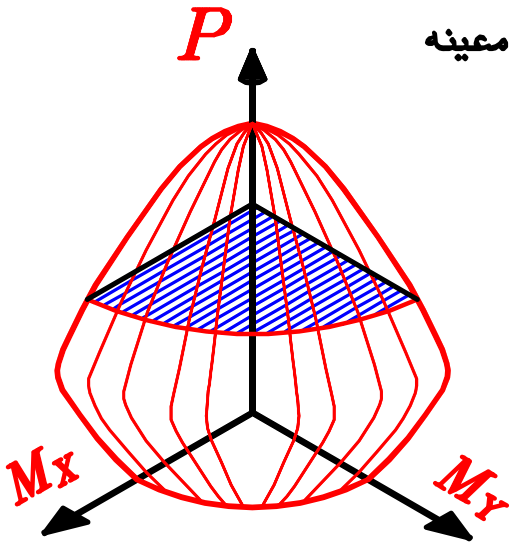
لكى نستطيع استخدام ال (*I.D.*) لتصميم قطاعات ال ***Bi-Axial Moment***

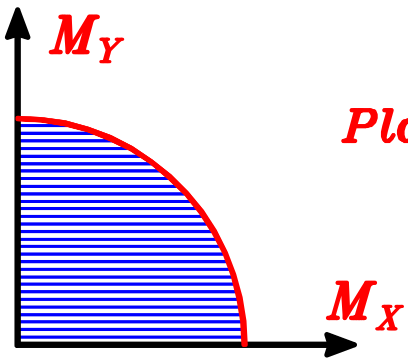
يتم قطع ال (*I.D.*) *3-D* بمستويات أفقيه أى مع كل تغير لقيمه *P* و تسمى ***Load Levels***



بحيث عند قيمه *P* معينه اى عند ***Load Level*** معينه

مع أخذ مقطع أفقى سيكون سيكون شكله



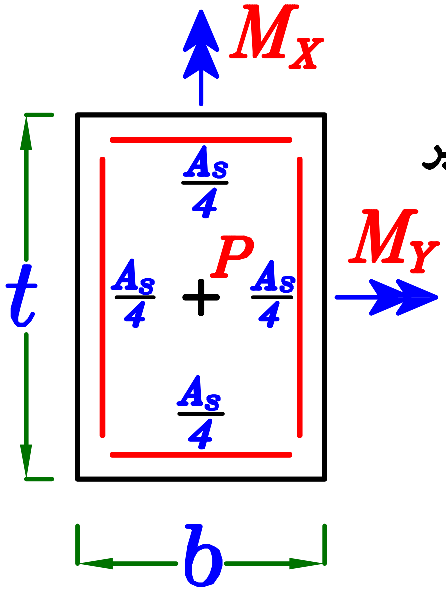


لتصميم قطاع *Bi-Axial Moment* بال *Inter Action Diagram*

يوجد حالتان .

1- Symmetrical RFT.

و فيها يتم تقسيم التسليح الكلى على الاربعه جهات بالتساوى .



و يفضل ان نستخدم هذه الحالة عندما يكون :

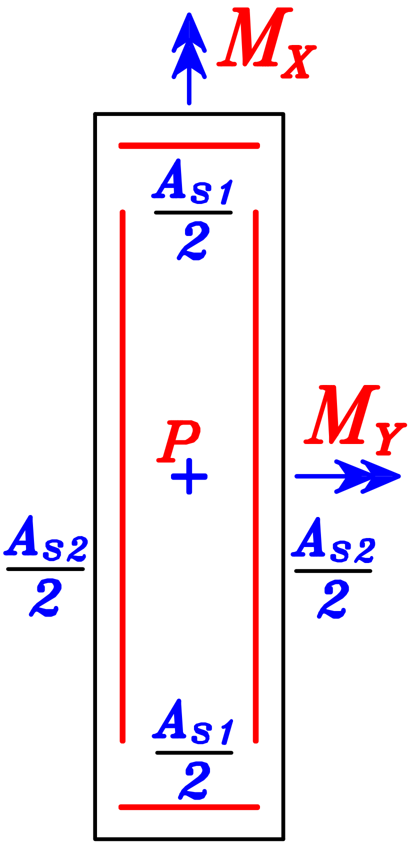
و عندما يكون العرض الكبير يقاوم ال *moment* الكبير

و العرض الصغير يقاوم ال *moment* الصغير

او عندما

Load Level $\boxed{R_b = \frac{P}{F_{cu}\, b\, t}} \geqslant 0.5$

2- Unsymmetrical RFT.



و فيها يتم حساب كميه تسليح كل *moment*

على حده و تقسيم هذا التسليح الى نصفين

و يفضل ان نستخدم هذه الحالة عندما يكون :

عندما

Load Level $\boxed{R_b = \frac{P}{F_{cu}\, b\, t}} \leqslant 0.5$

و عندما يكون العرض الكبير لا يقاوم ال *moment* الكبير

أو عندما يكون الفرق كبير بين طول و عرض القطاع

فلا يكون من المناسب ان نضع التسليح متساوى فى الاربع جهات .

1 – Symmetrical RFT.

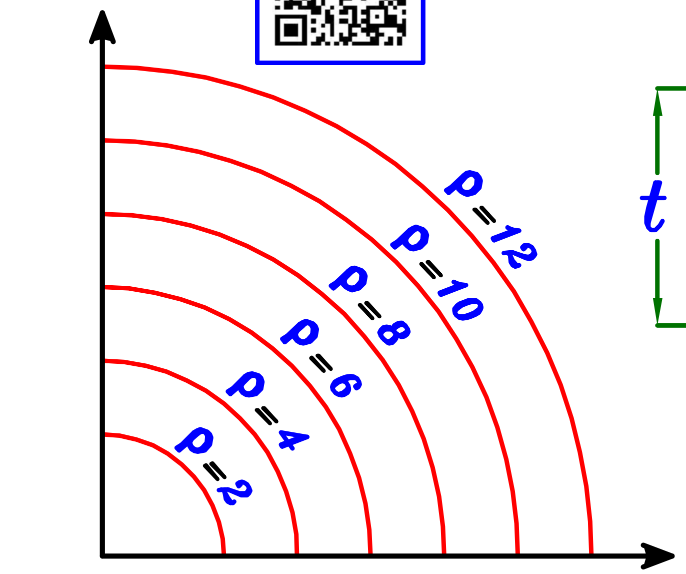
و يوجد طريقتان لتصميم القطاع الـ *Biaxial* و يكون *Symmetrical RFT.*

1 – Use *Biaxial I.D.*

2 – Use *Uniaxial I.D.*

Design using (*Biaxial Bending Interaction Diagram*)
(*Symmetrical arrangement of reinforcement*)

$$\frac{M_Y}{F_{cu}\, t\, b^2}$$



$$\frac{M_X}{F_{cu}\, b\, t^2}$$

Use ECCS Page (5-9) → (5-24)

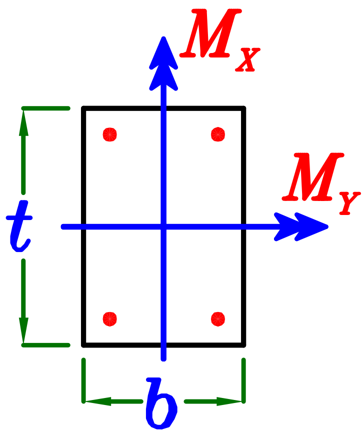


Chart Key
$F_y = \checkmark$
$R_b = \frac{P}{F_{cu}\, b\, t}$
$\zeta = \checkmark$

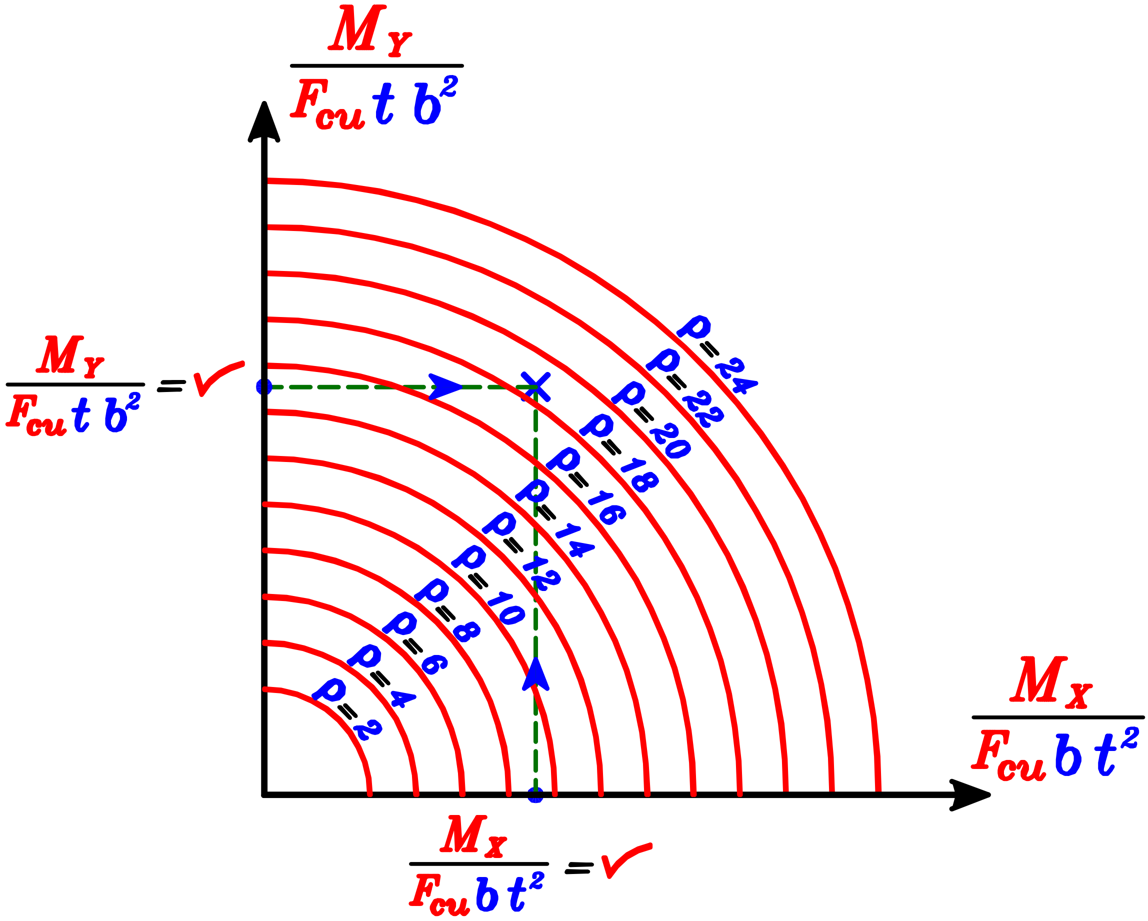
لتحديد أى *Chart* سيستخدم نحدد قيمه كل من F_y, R_b, ζ

$$R_b = \frac{P}{F_{cu}\, b\, t}$$

$$\zeta = \frac{t - 2\,Cover}{t} = 0.9$$ لانها القيمه الوحيده الموجوده فى الجداول

بعد تحديد الـ *Curve* بمعرفه كل من F_y, R_b, ζ

نحدد قيمه كل من $\boxed{\frac{M_Y}{F_{cu}\, t\, b^2}}$, $\boxed{\frac{M_X}{F_{cu}\, b\, t^2}}$



ثم نحدد قيمة ρ كما هو موضح

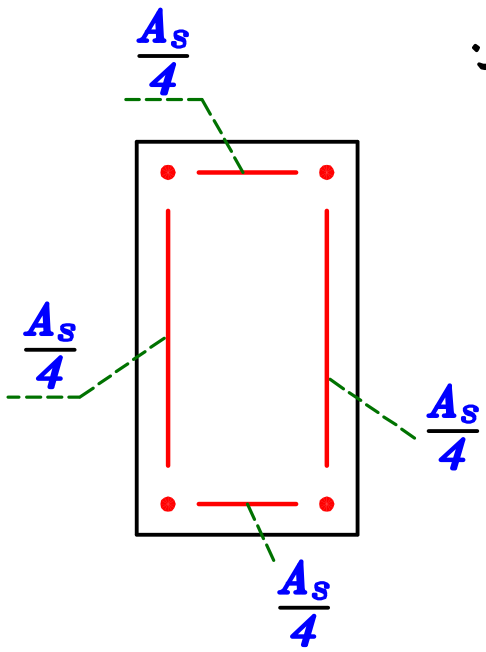
ثم نعوض فى المعادلات الأتيه لتحديد قيمه $A_{s\,total}$

$$\mu = \rho * F_{cu} * 10^{-4}$$

$$A_{s\,total} = \mu * b * t$$

$$A_{s\,min} = \frac{0.8}{100} * b * t$$

نقارن $A_{s\,total}$ بال $A_{s\,min}$ و نضع القيمه الاكبر.

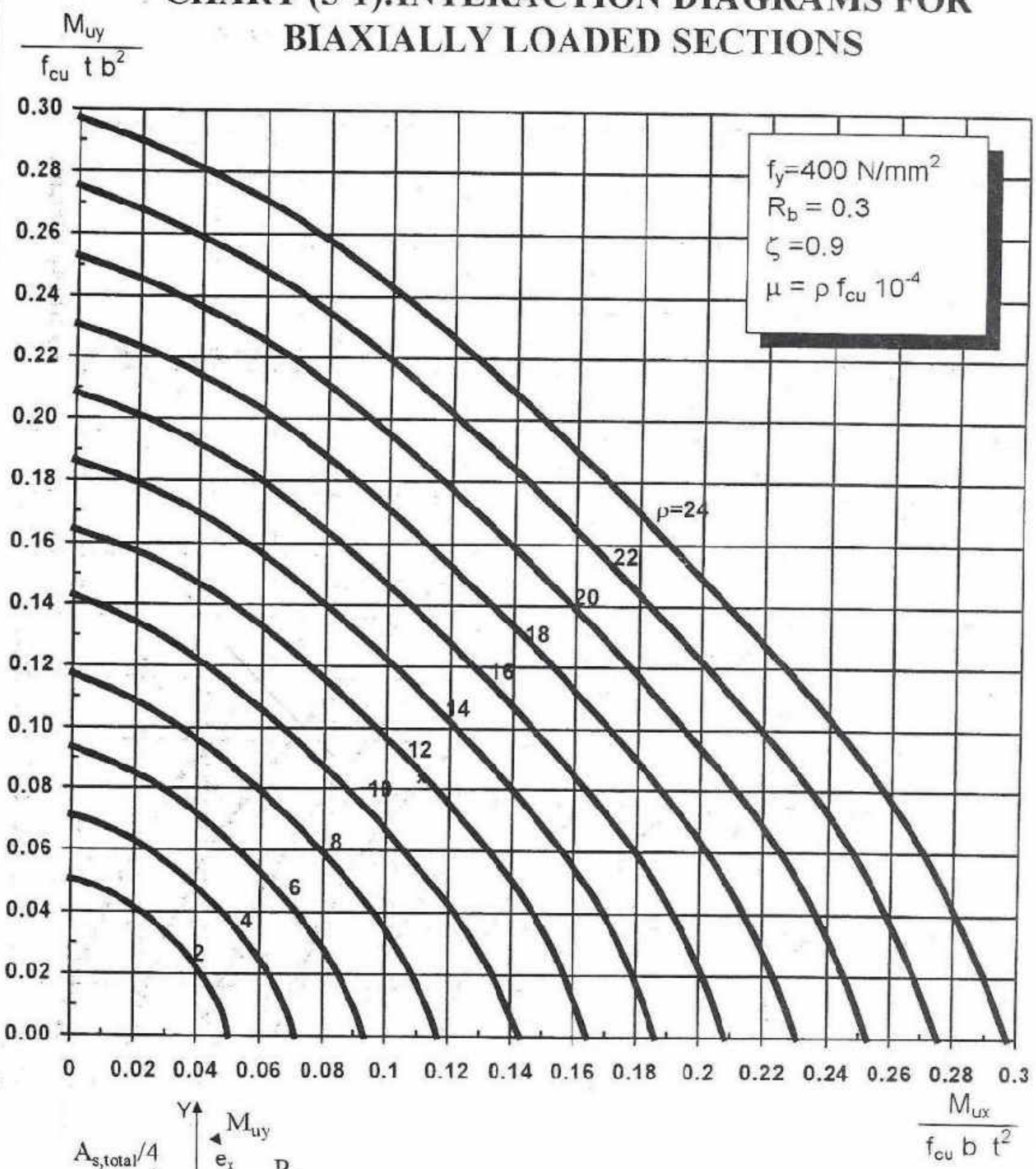


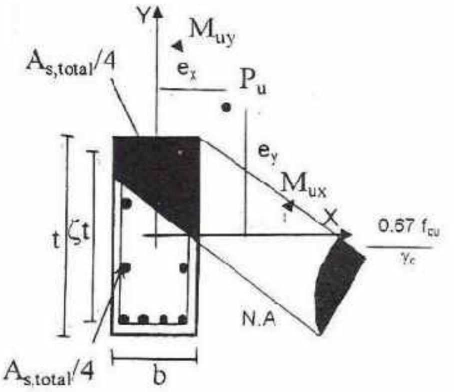
و يجب أن يكون عدد الاسياخ يقبل القسمه على ٤

نضع أربع أسياخ فى الاركان

ثم يقسم باقى الحديد بالتساوى على الاربع جهات

CHART (5-1): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS



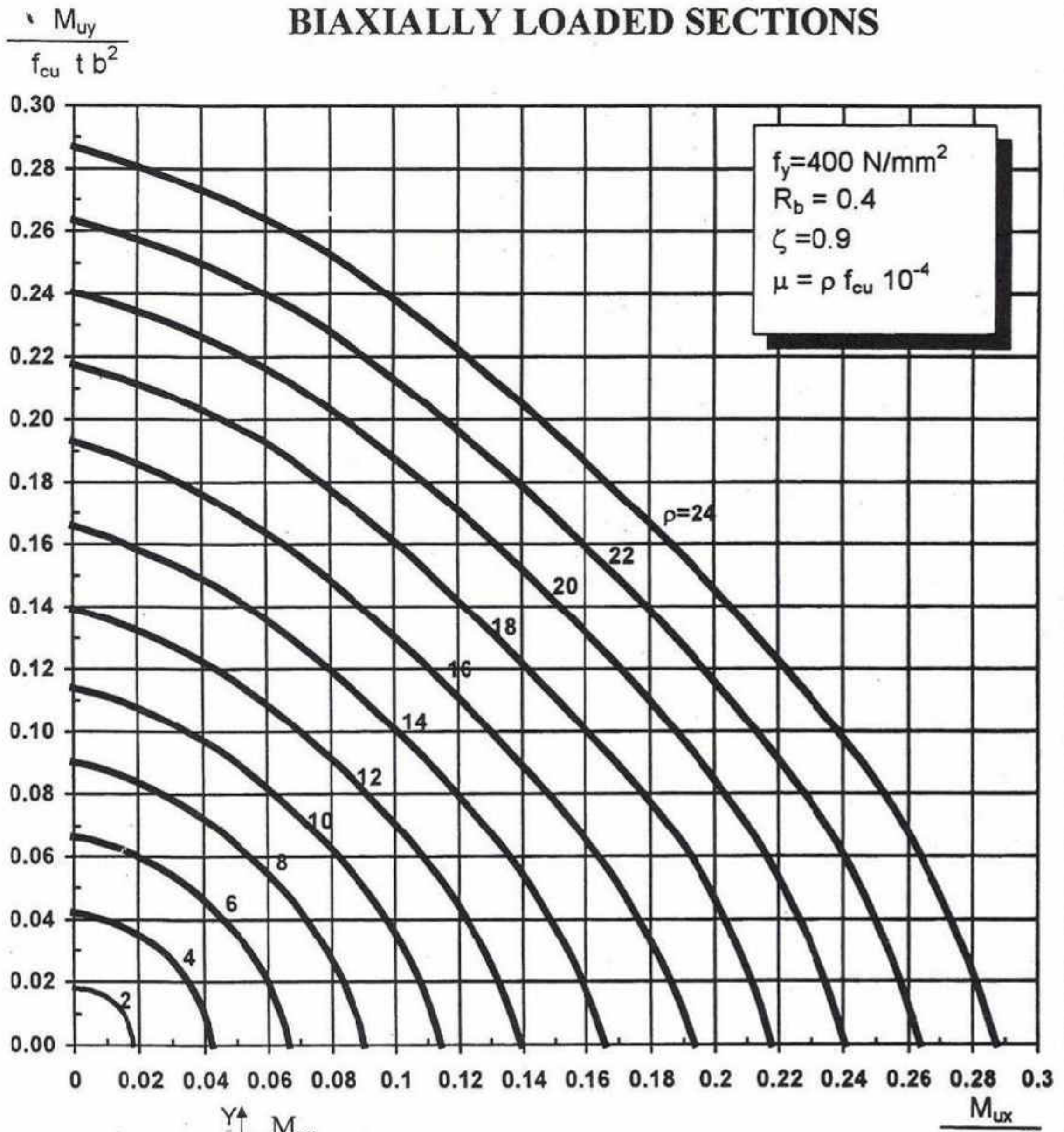


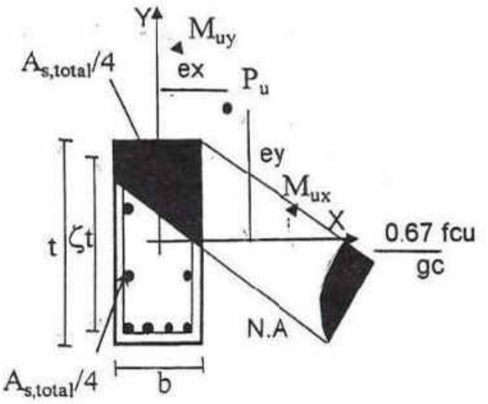
$$R_b = \frac{P_u}{f_{cu}\, b\, t}$$

$$\mu = \rho \ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu \ b \ t$$

CHART (5-2): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS





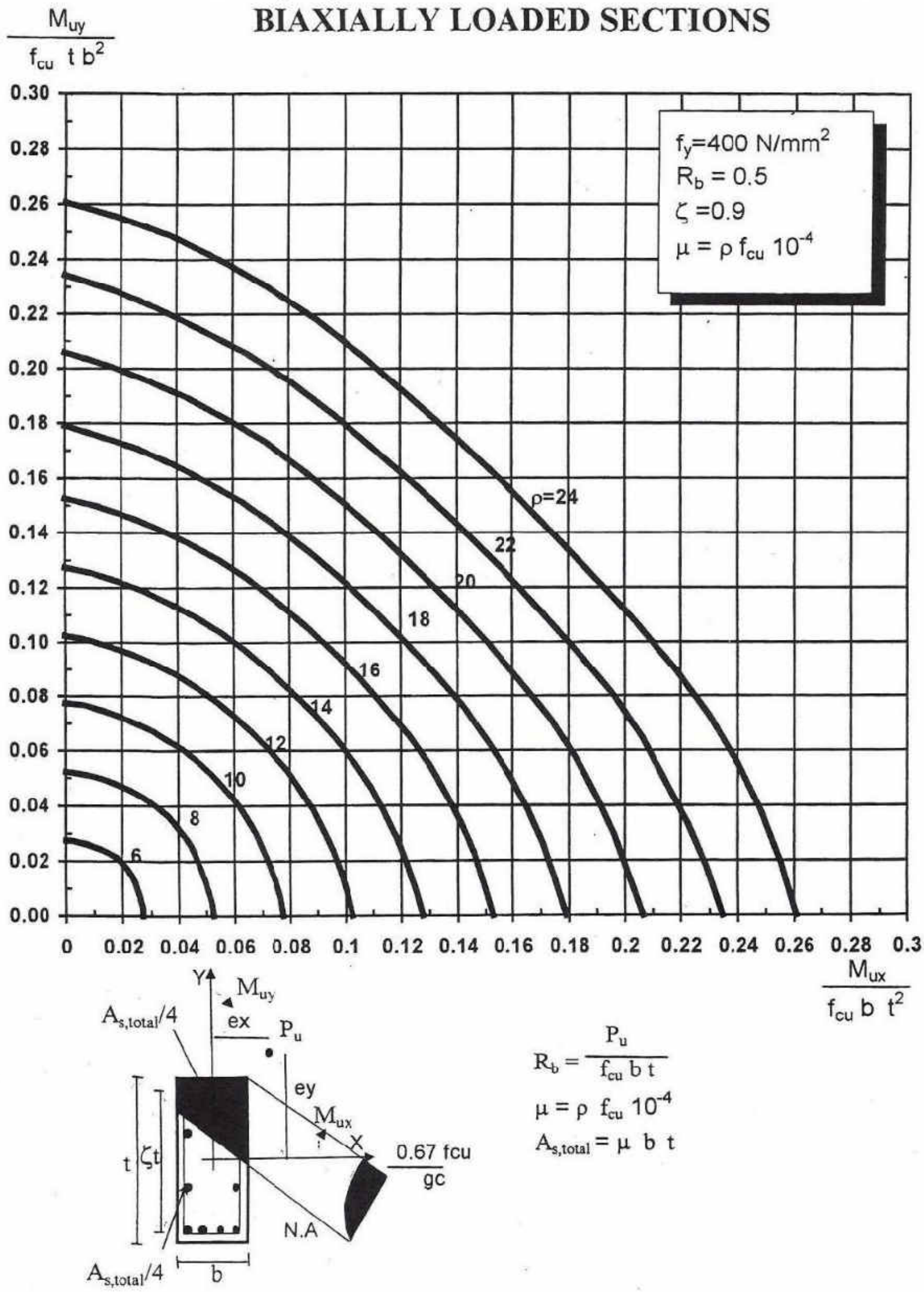
$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

ECCS 203-2001 Design Aids | Biaxial Bending

CHART (5-3):INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS

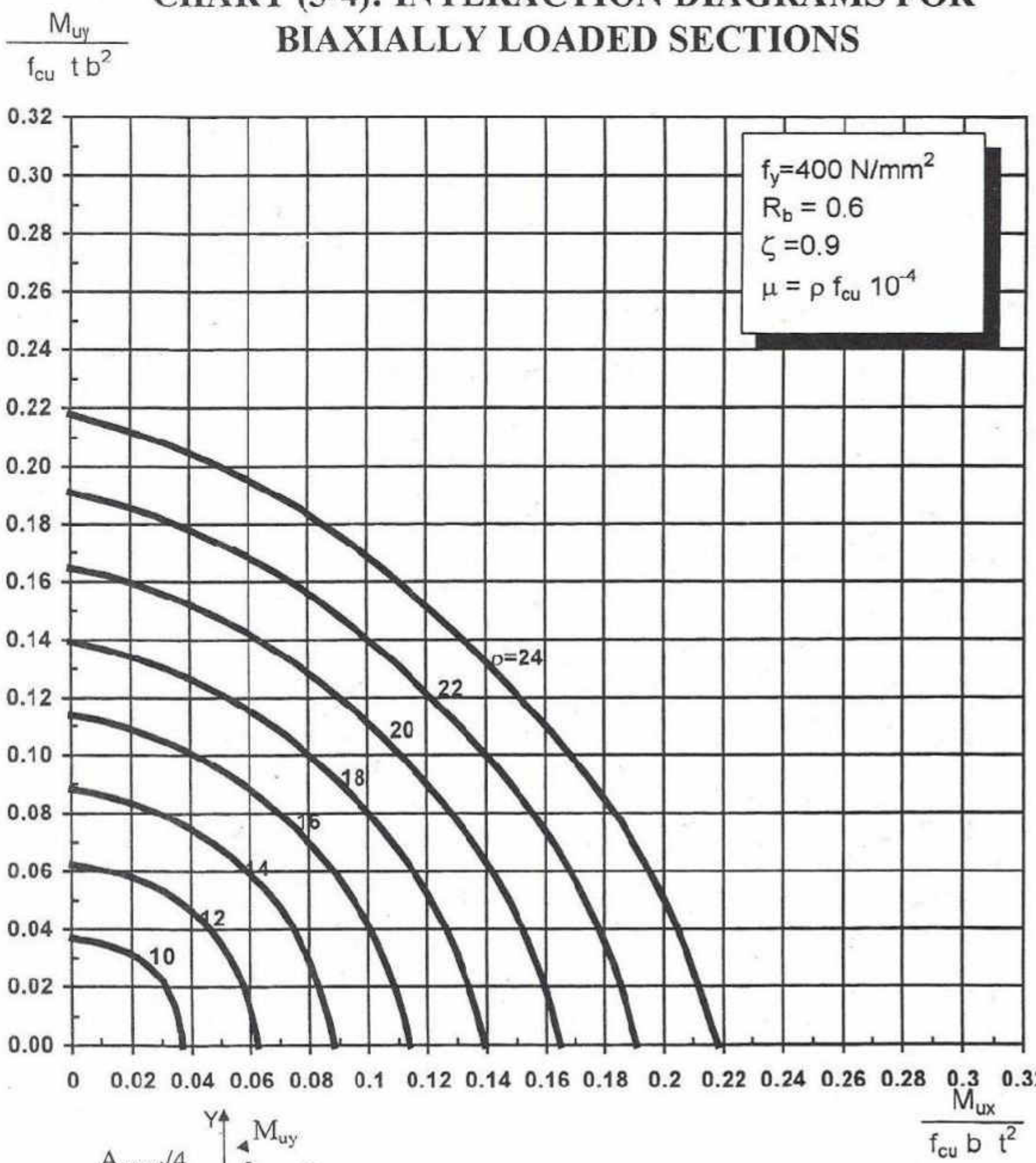


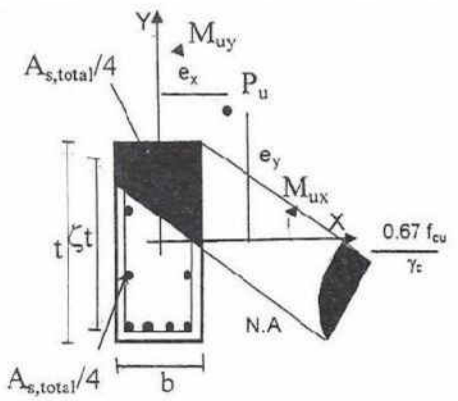
$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

CHART (5-4): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS





$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

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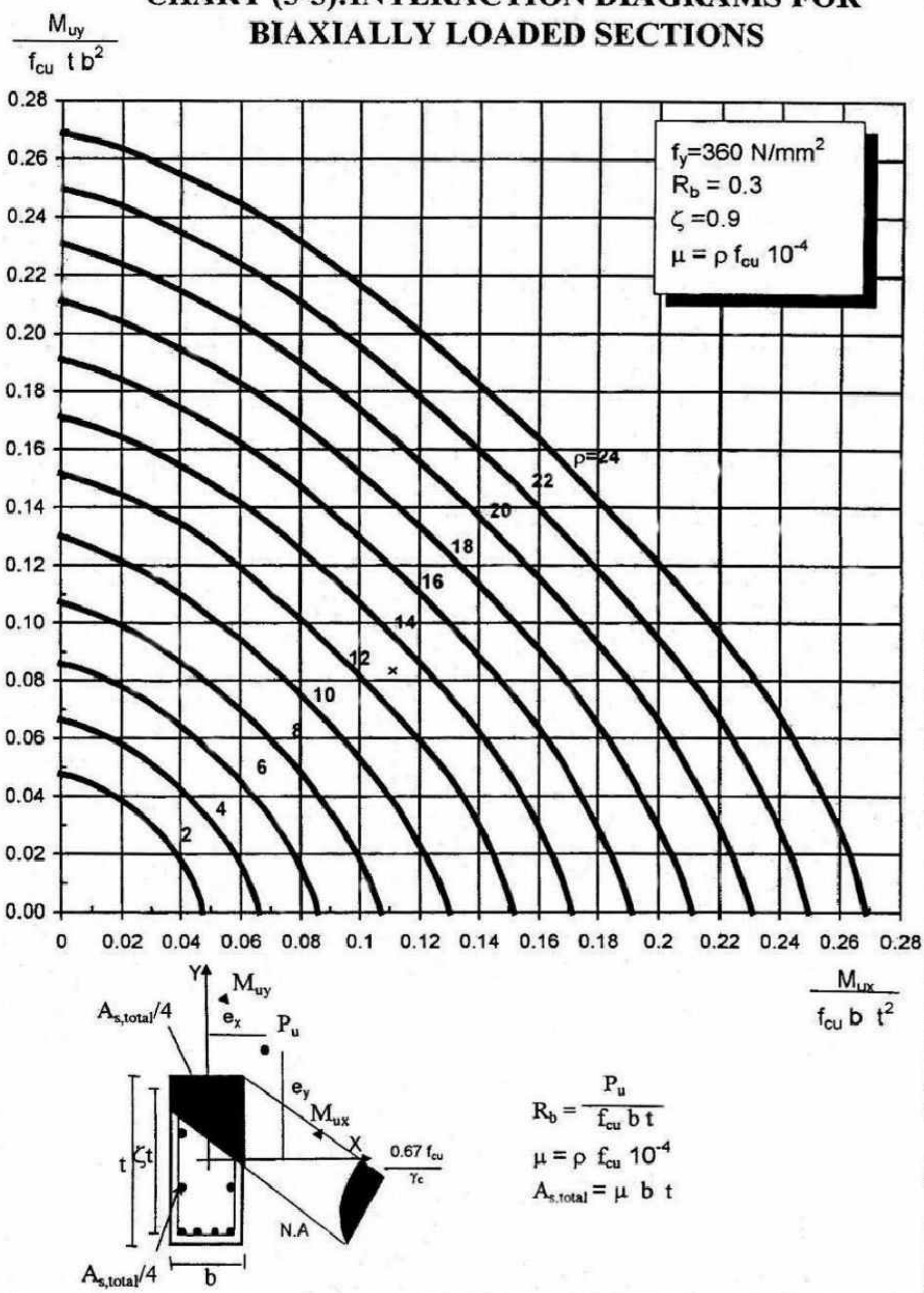
CHART (5-5): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS

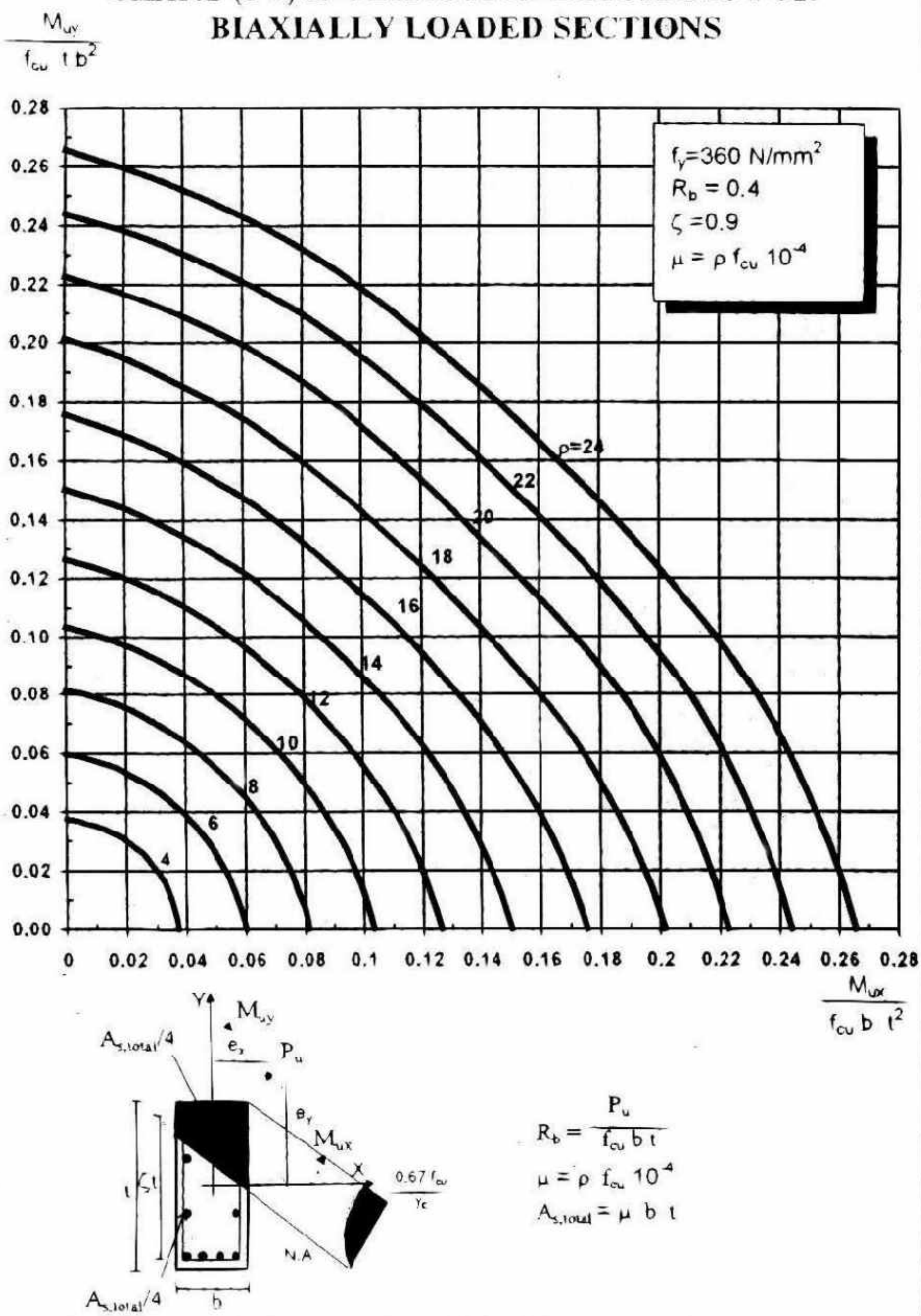


$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

CHART (5-6): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS



$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

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5-14

CHART (5-7):INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS

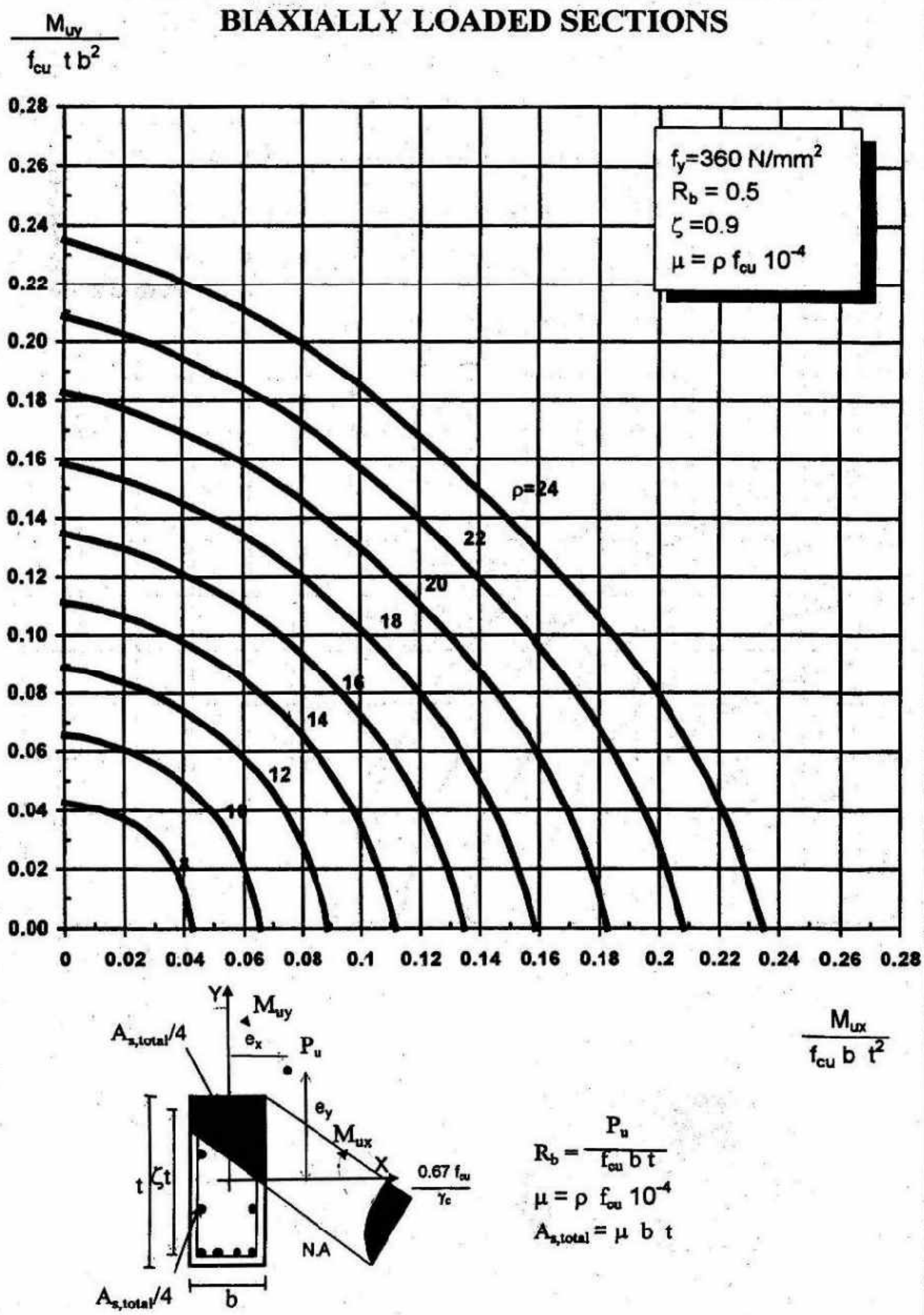

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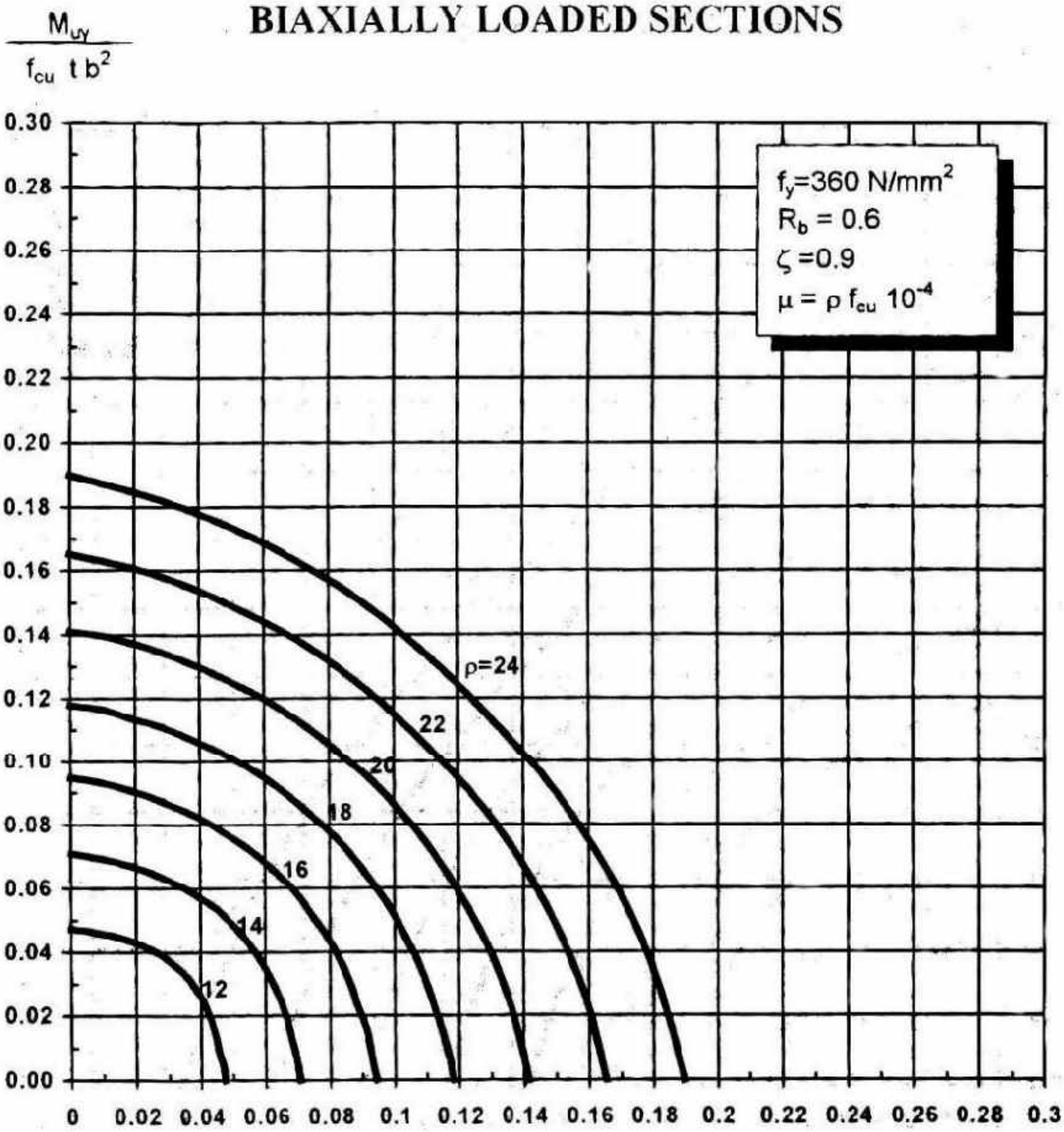
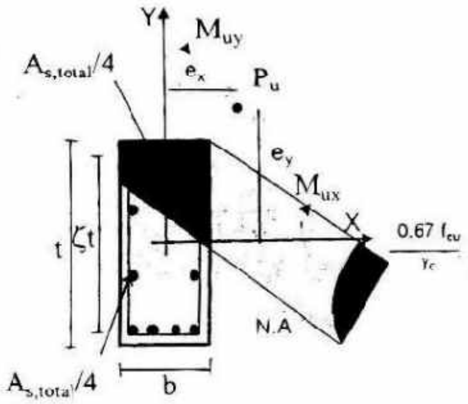
CHART (5-8): INTERACTION DIAGRAMS FOR BIAXIALLY LOADED SECTIONS





$$R_b = \frac{P_u}{f_{cu}\ b\ t}$$

$$\mu = \rho\ f_{cu}\ 10^{-4}$$

$$A_{s,total} = \mu\ b\ t$$

Example.

Data:

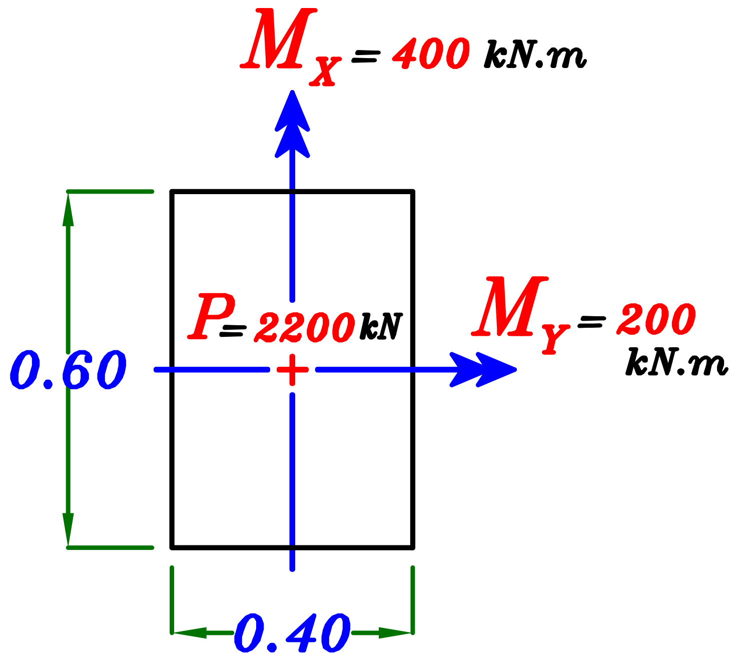
$F_{cu} = 25 \ N\backslash mm^2$

$F_y = 360 \ N\backslash mm^2$

$P_{U.L.} = 2200 \ kN$

$M_X \ (U.L.) = 400 \ kN.m$

$M_Y \ (U.L.) = 200 \ kN.m$



Req:

Design the Section.

---

assume $\zeta = 0.90$ ---------- ECCS لا توجد قيمه غيرها فى ال

$$R_b = \frac{P}{F_{cu}\, b\, t} = \frac{2200 * 10^3}{25 * 400 * 600} = 0.366 \longrightarrow \text{Not in ECCS}$$

لانه لا توجد قيمه لـ $R_b = 0.366$ فى كتاب ECCS فيتم حساب قيمتين لـ ρ
مره عند $R_b = 0.30$ و مره عند $R_b = 0.40$ ثم أخذ قيمه للـ ρ بينهم

For $R_b = 0.30 \longrightarrow$ $\boxed{\text{ECCS Page (5-13)}}$

$$\frac{M_X}{F_{cu}\, b\, t^2} = \frac{400 * 10^6}{25 * 400 * 600^2} = 0.111$$

$$\frac{M_Y}{F_{cu}\, t\, b^2} = \frac{200 * 10^6}{25 * 600 * 400^2} = 0.083$$

$\rho = 12.8$

For $R_b = 0.40 \longrightarrow$ $\boxed{\text{ECCS Page (5-14)}}$

$$\frac{M_X}{F_{cu}\, b\, t^2} = \frac{400 * 10^6}{25 * 400 * 600^2} = 0.111$$

$$\frac{M_Y}{F_{cu}\, t\, b^2} = \frac{200 * 10^6}{25 * 600 * 400^2} = 0.083$$

$\rho = 15$

نأخذ ρ قيمه بينهم

To get value of ρ For $R_b = 0.366$

$R_b = 0.30 \longrightarrow \rho = 12.8$

$R_b = 0.40 \longrightarrow \rho = 15$

$\rho = 13.9$ نأخذ قيمه بينهم

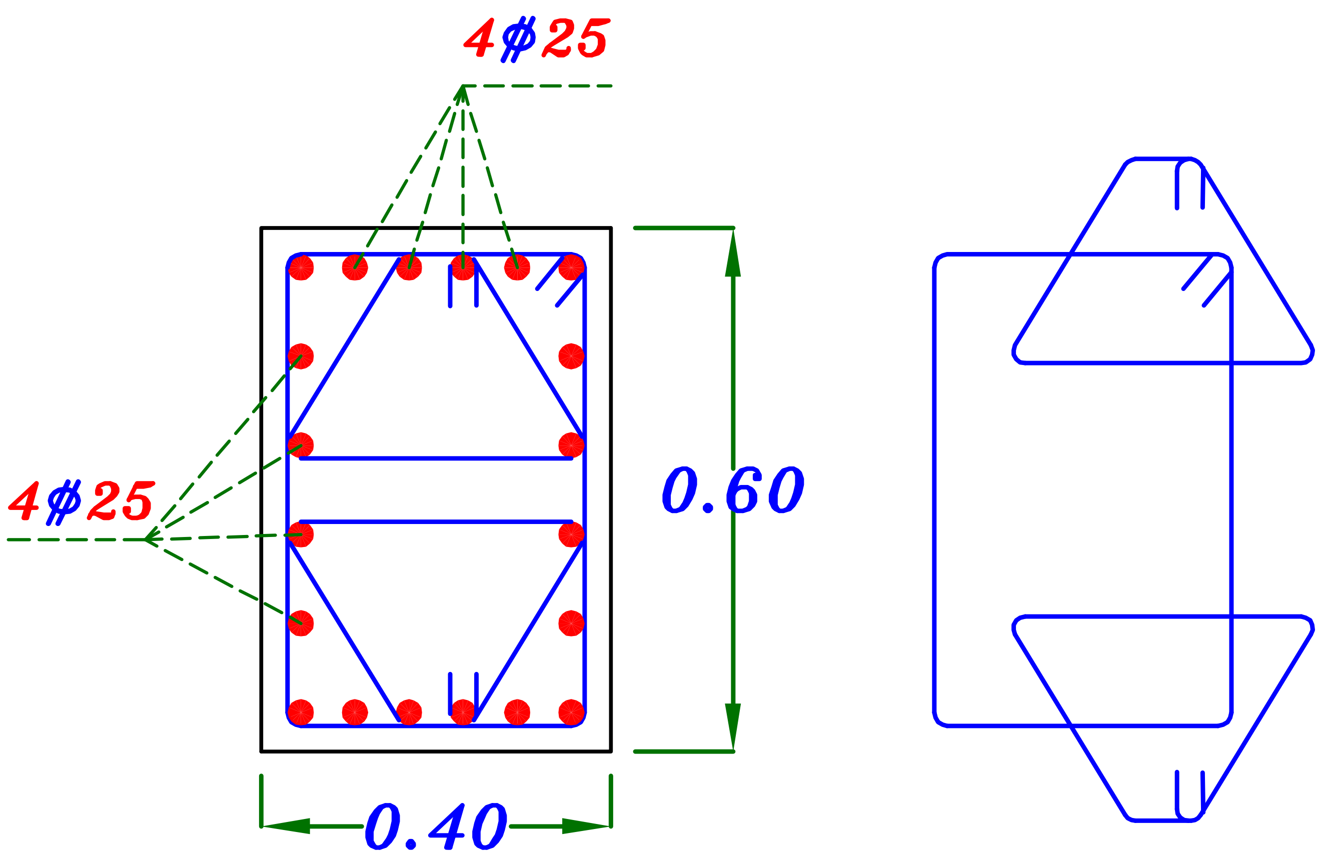
$$\mu = \rho * F_{cu} * 10^{-4} = 13.9 * 25 * 10^{-4} = 0.03475$$

$$A_{s\,total} = \mu * b * t = 0.03475 * 400 * 600 = 8340 \ mm^2$$

– Check $A_{s\,min.} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 400 * 600 = 1920 \ mm^2$

$$A_s = A_{s\,total} = 8340 \, mm^2$$

$20 \phi 25$



Symmetrical RFT.

2- Design using (Uniaxial Bending Interaction Diagram)
(Symmetrical arrangement of reinforcement)

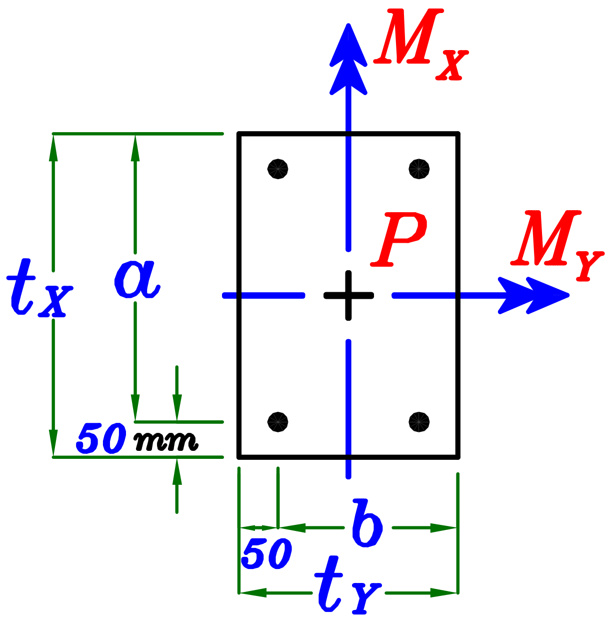
طريقه أخرى تعتمد على تحويل تأثير العزمين الى عزم واحد فقط مكافئ لهم.

نحدد قيمه d التى تقاوم M_X و تسمى مثلا a

$$\boxed{a = t_X - 50\ mm}$$

نحدد قيمه d التى تقاوم M_Y و تسمى مثلا b

$$\boxed{b = t_Y - 50\ mm}$$



نحدد العزم الذى سيكون تأثيره اقل على القطاع و نهمله و نأخذ العزم الذى تأثيره اكبر على القطاع و نعمل على تكبيره لكن يكون مكافئ للعزمين معا .

و لمعرفه اى عزم سيتم اهماله و ايهم سيتم تكبيره نحسب نسبه كل عزم على ال d التى ستقاومه .

Calculate $\quad \dfrac{M_X}{a} \; , \; \dfrac{M_Y}{b}$

We have two cases:

① IF $\dfrac{M_X}{a} > \dfrac{M_Y}{b} \xrightarrow{\text{Neglect}} M_Y \xrightarrow{\text{And Calculate}} M_X`$

② IF $\dfrac{M_Y}{b} > \dfrac{M_X}{a} \xrightarrow{\text{Neglect}} M_X \xrightarrow{\text{And Calculate}} M_Y`$

① IF $\frac{M_X}{a} > \frac{M_Y}{b}$ $\xrightarrow{\text{Neglect}}$ M_Y $\xrightarrow{\text{And Calculate}}$ $M_{X`}$

Where: $$\boxed{M_{X`} = M_X + \beta \frac{a}{b} M_Y}$$

$$\boxed{\beta = 0.9 - \frac{R_b}{2}}$$ --- $0.6 \leqslant \beta \leqslant 0.8$

IF $\beta < 0.6 \longrightarrow$ Take $\beta = 0.6$

IF $\beta > 0.8 \longrightarrow$ Take $\beta = 0.8$

Where R_b is the Load Level $R_b = \frac{P}{F_{cu}\, b\, t}$

Or we can use table in Code Page (6–59)

R_b	$\leqslant 0.2$	0.3	0.4	0.5	$\geqslant 0.6$
β	0.80	0.75	0.70	0.65	0.60

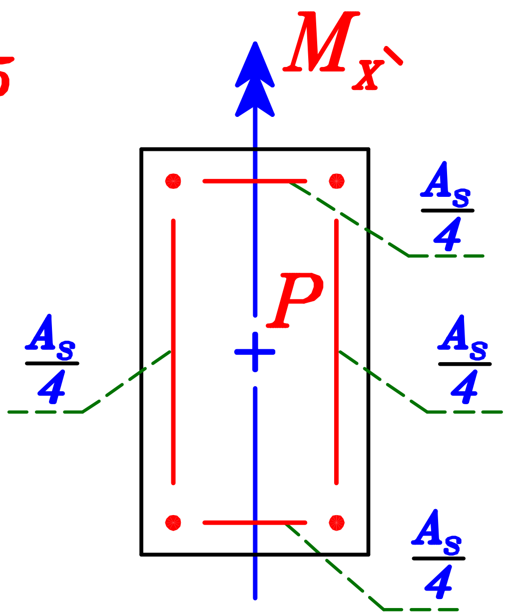
design the section on $P, M_{X`}$

Using Uniaxial I.D. even IF $\frac{e}{t} > 0.5$

Then get $A_s = A_{s`}$

$A_{s\,total} = A_s + A_{s`}$

Check $A_{s\,total}$ with $\boxed{A_{s\,min} = \frac{0.8}{100} * b * t}$



نضع أربع أسياخ فى الاركان

ثم يقسم باقى الحديد بالتساوى على الاربع جهات

② IF $\frac{M_Y}{b} > \frac{M_X}{a}$ $\xrightarrow{\text{Neglect}}$ M_X $\xrightarrow{\text{And Calculate}}$ $M_{Y}^{`}$

Where: $$\boxed{M_{Y}^{`} = M_X + \beta \, \frac{b}{a} \, M_X}$$

β is the same as before.

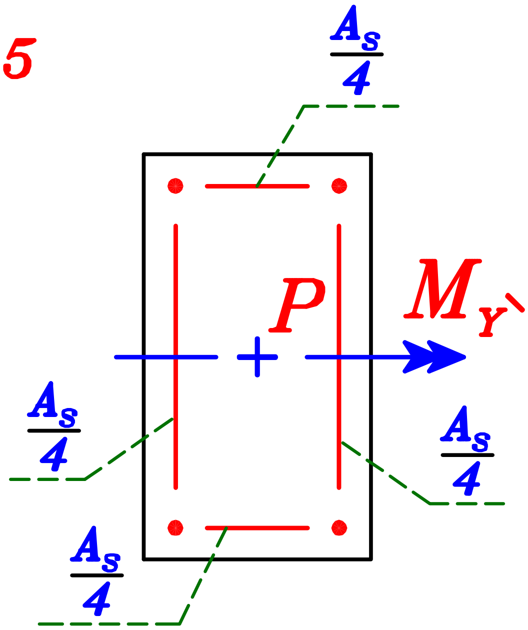
design the section on $P, M_{X}^{`}$

Using Uniaxial I.D. even IF $\frac{e}{t} > 0.5$

Then get $A_s = A_s^{`}$

$A_{s\,total} = A_s + A_s^{`}$

Check $A_{s\,total}$ with $\boxed{A_{s\,min} = \frac{0.8}{100} * b * t}$



نضع أربع أسياخ فى الاركان

Example.

Data:

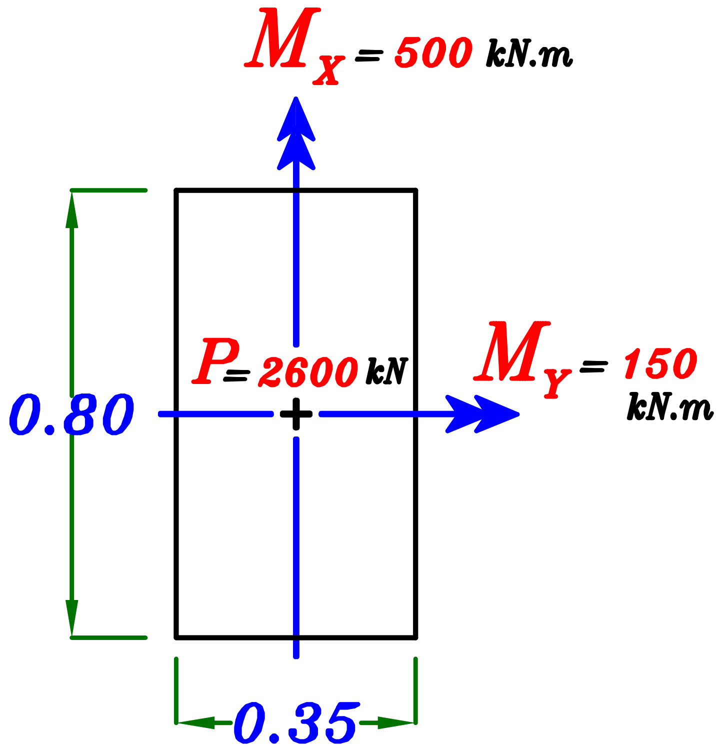
$F_{cu} = 30 \quad N\backslash mm^2$

$F_y = 360 \quad N\backslash mm^2$

$P_{U.L.} = 2600 \ kN$

$M_X \ (U.L.) = 500 \ kN.m$

$M_Y \ (U.L.) = 150 \ kN.m$



Req:

Design the Section with symmetric RFT.

$a = t_X - 50 \ mm = 800 - 50 = 750 \ mm = 0.75 \ m$

$b = t_Y - 50 \ mm = 350 - 50 = 300 \ mm = 0.30 \ m$

$\frac{M_X}{a} = \frac{500}{0.75} = 666.6 \quad , \quad \frac{M_Y}{b} = \frac{150}{0.30} = 500$

$\frac{M_X}{a} > \frac{M_Y}{b} \longrightarrow$ Neglect M_Y and design the Sec. on $M_X`$

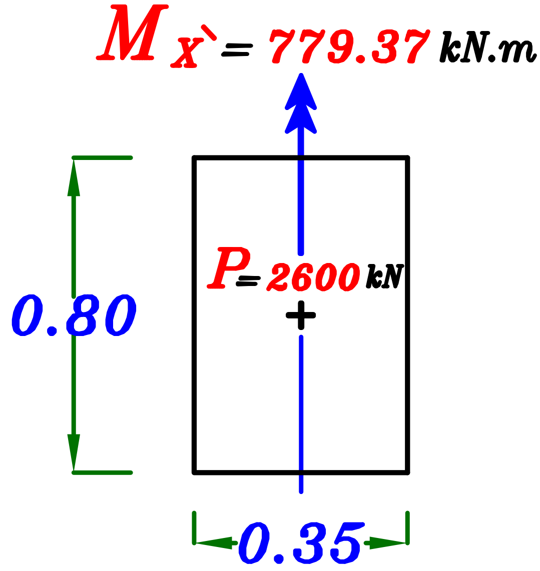
$$R_b = \frac{P}{F_{cu}\, b\, t} = \frac{2600 * 10^3}{30 * 350 * 800} = 0.31$$

$$\beta = 0.9 - \frac{R_b}{2} = 0.9 - \frac{0.31}{2} = 0.745 \begin{array}{l} > 0.6 \\ < 0.8 \end{array}$$

$$M_X` = M_X + \beta \left(\frac{a}{b}\right) M_Y$$

$$M_X` = 500 + 0.745 \left(\frac{0.75}{0.30}\right) 150 = 779.37 \ kN.m$$

Using Uniaxial I.D.

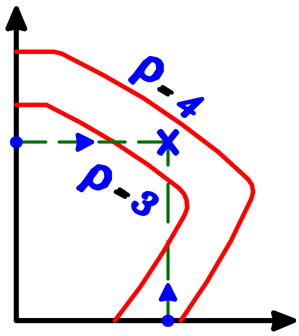


$$e = \frac{M}{P} = \frac{779.37}{2600} = 0.299 \text{ m}$$

$$\zeta = \frac{800-100}{800} = 0.87 = 0.80$$

use ECCS Design Aids Page 4–24

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{2600 * 10^3}{30*350*800} = 0.31 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{779.37 * 10^6}{30*350*800^2} = 0.116 \end{aligned}\right\} \rho = 3.6$$



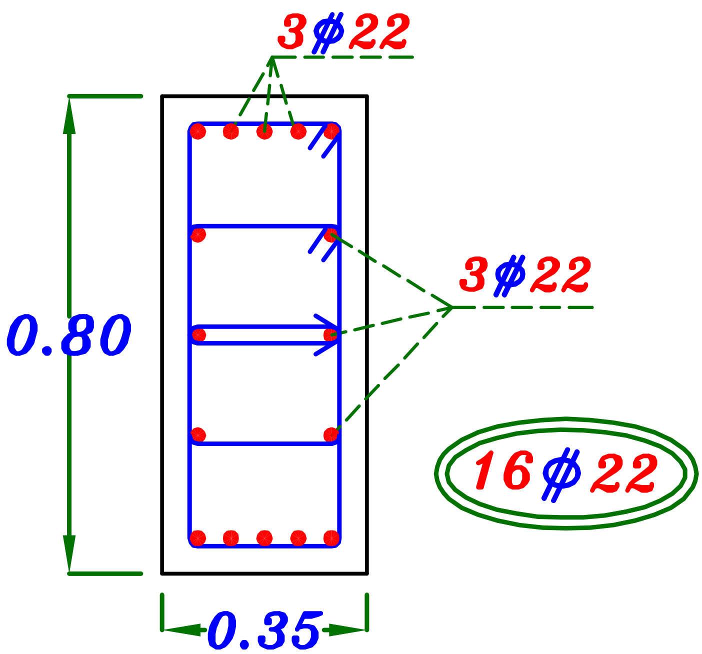
$$\mu = \rho * F_{cu} * 10^{-4} = 3.6 * 30 * 10^{-4} = 0.0108$$

$$A_s = A_{s`} = \mu * b * t = 0.0108 * 350 * 800 = 3024 \text{ mm}^2$$

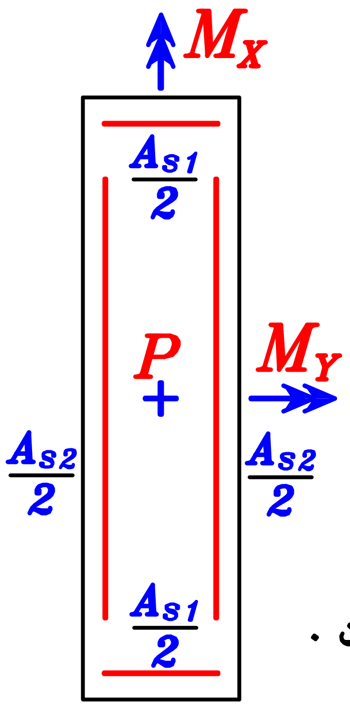
$$A_{s\,Total} = A_s + A_{s`} = 2 * 3024 = 6048 \text{ mm}^2$$

$$A_{s_{min}} = \frac{0.80}{100} * b * t = \frac{0.80}{100} * 350 * 800 = 2240 \text{ mm}^2 < A_{s_{total}}$$

Take $A_s = A_{s\,Total} = 6048 \text{ mm}^2$ 16ϕ22



2 – Unsymmetrical RFT.



و فيها يتم حساب كميه تسليح كل moment على حده و تقسيم هذا التسليح الى نصفين

و ممكن ان نستخدم هذه الحاله عندما يكون :

Load Level $\boxed{R_b = \frac{P}{F_{cu}\, b\, t}} \leqslant 0.5$

و عندما يكون العرض الكبير لا يقاوم ال moment الكبير

أو عندما يكون الفرق كبير بين طول و عرض القطاع

فلا يكون من المناسب ان نضع التسليح متساوى فى الاربع جهات .

تعتمد هذه الطريقه على ضرب قيمه كلا من M_X و M_Y فى معامل α_b

$$\boxed{M_X` = \alpha_b * M_X} \quad , \quad \boxed{M_Y` = \alpha_b * M_Y}$$

To calculate α_b

– Calculate $R_b = \frac{P}{F_{cu}\, b\, t}$ يجب ان لاتزيد قيمه R_b عن 0.5

– Calculate the Ratio $\frac{M_X \backslash a}{M_Y \backslash b}$

– Calculate α_b From Table at Code page 6–61

R_b \ $\frac{M_X \backslash a}{M_Y \backslash b}$	∞	3.0	2.0	1.0	0.5	0.33	Zero
$R_b \leqslant 0.1$	1.0	1.20	1.25	1.30	1.25	1.20	1.0
$R_b = 0.2$	1.0	1.35	1.50	1.75	1.50	1.35	1.0
$R_b = 0.3$	1.0	1.25	1.35	1.40	1.35	1.25	1.0
$R_b = 0.4$	1.0	0.95	0.95	0.95	0.95	0.95	1.0
$R_b \geqslant 0.5$	1.0	0.65	0.70	0.75	0.70	0.65	1.0

Uniaxial I.D. ثم يتم تصميم القطاع مرتين باستخدام

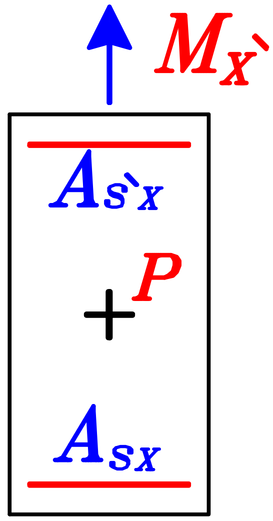
① **Design on** $P, M_{X`}$

Using *Uniaxial I.D.*

Get $A_{sx} = A_{s`x}$

ثم يتم وضع التسليح $A_{sx} + A_{s`x}$

فى الاتجاه الرأسى لمقاومه $M_{X`}$



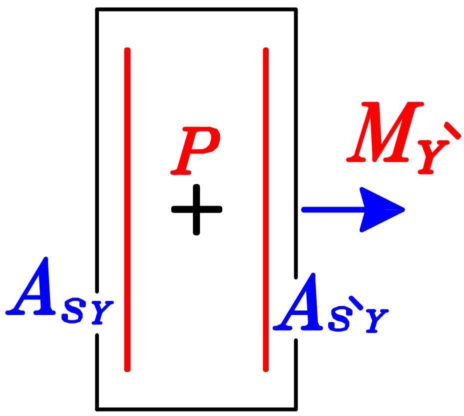
② **Design on** $P, M_{Y`}$

Using *Uniaxial I.D.*

Get $A_{sY} = A_{s`Y}$

ثم يتم وضع التسليح $A_{sY} + A_{s`Y}$

فى الاتجاه الافقى لمقاومه $M_{Y`}$



Check $A_{s\,min}$

يتم حساب $A_{sT} = A_{sx} + A_{s`x} + A_{sY} + A_{s`Y}$

و حساب $A_{s\,min} = \frac{0.80}{100} * b * t$

$$IF \quad A_{sT} > A_{s\,min} \xrightarrow{use} A_{sx},\ A_{s`x},\ A_{sY}\ \&\ A_{s`Y}$$

$$IF \quad A_{sT} < A_{s\,min} \xrightarrow{use} A_{s\,min}$$

يتم تقسيم قيمه A_s الكليه التى ستوضع فى القطاع على الاربع اتجاهات لكن بنسبه

$$\text{نسبه } \frac{M_X \text{ مساحه الحديد الذى سيوضع فى اتجاه}}{M_Y \text{ مساحه الحديد الذى سيوضع فى اتجاه}} = \text{نسبه } \frac{A_{SX}}{A_{SY}}$$

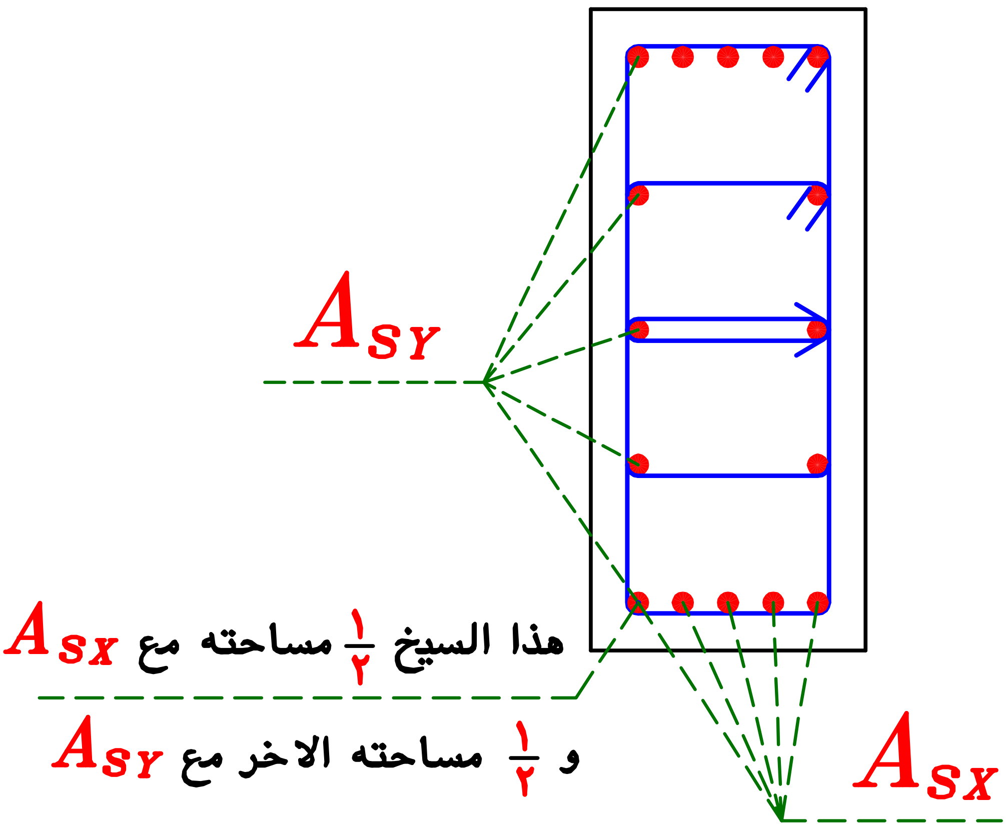
مع اخذ السيخ الذى سيوضع فى الركن نصف مساحته مع A_{SX} و النصف الاخر مع A_{SY}

$$\text{No. of Bars to resist } M_X = \frac{A_{SX}}{A_{SX}+A_{SY}} * \text{Total No. of bars}$$

و يتم تقسيم الحديد الى نصفين نصف اسفل القطاع و نصف اعلى

$$\text{No. of Bars to resist } M_Y = \frac{A_{SY}}{A_{SX}+A_{SY}} * \text{Total No. of bars}$$

و يتم تقسيم الحديد الى نصفين نصف جهه اليمين و نصف جهه اليسار .



Example.

Data:

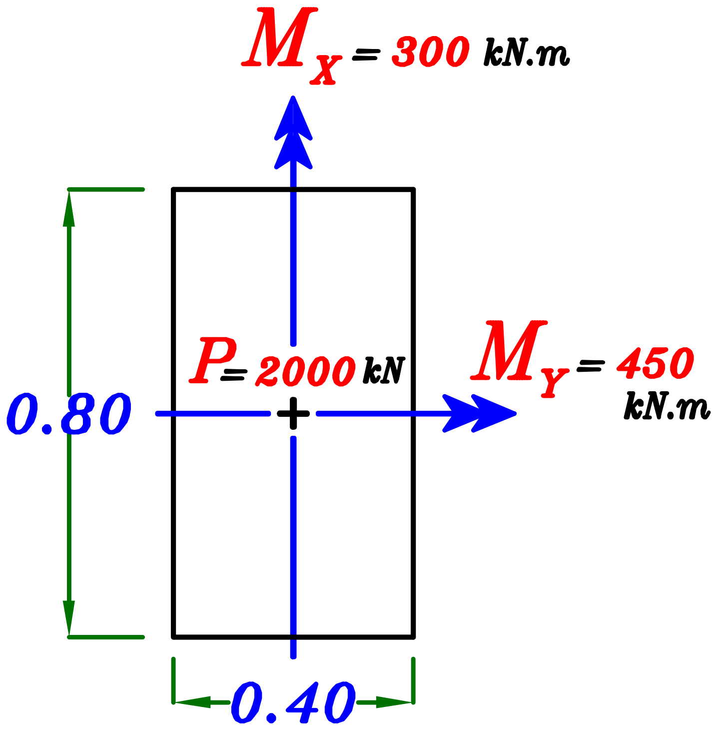
$F_{cu} = 30 \quad N\backslash mm^2$

$F_y = 360 \quad N\backslash mm^2$

$P_{U.L.} = 2000 \quad kN$

$M_X \,(U.L.) = 300 \quad kN.m$

$M_Y \,(U.L.) = 450 \quad kN.m$



Req:

Design the Section with unsymmetric RFT.

$$R_b = \frac{P}{F_{cu}\, b\, t} = \frac{2000 * 10^3}{30 * 400 * 800} = 0.208 < 0.5 \quad \therefore o.k.$$

$$a = t_X - 50\ mm = 800 - 50 = 750\ mm = 0.75\ m$$

$$b = t_Y - 50\ mm = 400 - 50 = 350\ mm = 0.35\ m$$

$$\frac{M_X}{a} = \frac{300}{0.75} = 400 \quad , \quad \frac{M_Y}{b} = \frac{450}{0.30} = 1500$$

$$\therefore \frac{M_X\backslash a}{M_Y\backslash b} = \frac{400}{1500} = 0.267$$

Calculate α_b From Table at Code page 6–61

R_b \ $\frac{M_X\backslash a}{M_Y\backslash b}$	∞	3.0	2.0	1.0	0.5	0.33	Zero
$R_b \leqslant 0.1$	1.0	1.20	1.25	1.30	1.25	1.20	1.0
$R_b = 0.2$	1.0	1.35	1.50	1.75	1.50	1.35	1.0
$R_b = 0.3$	1.0	1.25	1.35	1.40	1.35	1.25	1.0
$R_b = 0.4$	1.0	0.95	0.95	0.95	0.95	0.95	1.0
$R_b \geqslant 0.5$	1.0	0.65	0.70	0.75	0.70	0.65	1.0

From Interpolation

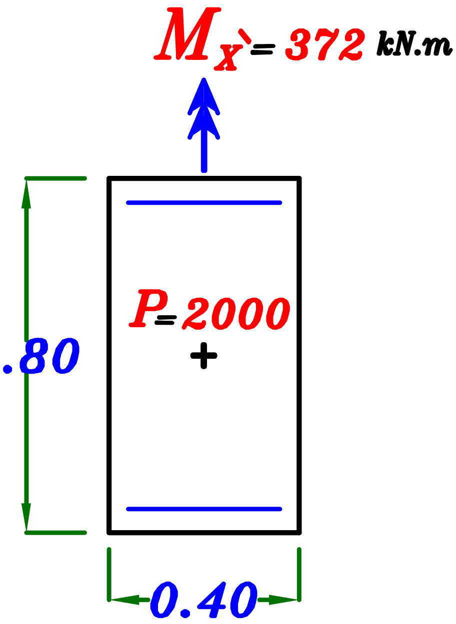
$$\boxed{\alpha_b = 1.24}$$

$$M_{X}` = \alpha_b * M_X = 1.24 * 300 = 372 \ kN.m$$

$$M_{Y}` = \alpha_b * M_Y = 1.24 * 450 = 558 \ kN.m$$

Uniaxial I.D. ثم يتم تصميم القطاع مرتين باستخدام

① Design on $P, M_{X}`$



$$\zeta = \frac{800-100}{800} = 0.87 = 0.80$$

use ECCS Design Aids Page 4–24

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{2000 * 10^3}{30 * 400 * 800} = 0.208$$

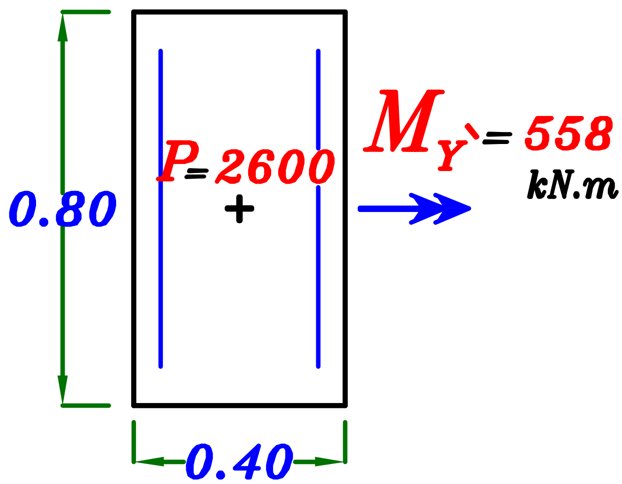
$$\frac{M_{X}`}{F_{cu}\, b\, t^2} = \frac{372 * 10^6}{30 * 400 * 800^2} = 0.048$$

$$\left.\begin{matrix} 0.208 \\ 0.048 \end{matrix}\right\} \rho < 1.0 \longrightarrow \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 0.003$$

$$A_{sx} = A_{s`x} = \mu * b * t = 0.003 * 400 * 800 = 960 \ mm^2$$

② Design on $P, M_{Y}`$



$$\zeta = \frac{400-100}{400} = 0.75 = 0.70$$

use ECCS Design Aids Page 4–25

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{2000 * 10^3}{30*800*400} = 0.208$$

$$\frac{M_{Y`}}{F_{cu}\, b\, t^2} = \frac{558 * 10^6}{30*800*400^2} = 0.145$$

$$\rho = 4.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 4.0 * 30 * 10^{-4} = 0.012$$

$$A_{sY} = A_{s`Y} = \mu * b * t = 0.012 * 800 * 400 = 3840 \ mm^2$$

Check $A_{s\,min}$

$$A_{sT} = A_{sX} + A_{s`X} + A_{sY} + A_{s`Y} = 2*960 + 2*3840 = 9600 \ mm^2$$

$$A_{s\,min} = \frac{0.80}{100} * b * t = \frac{0.80}{100} * 400 * 800 = 2560 \ mm^2$$

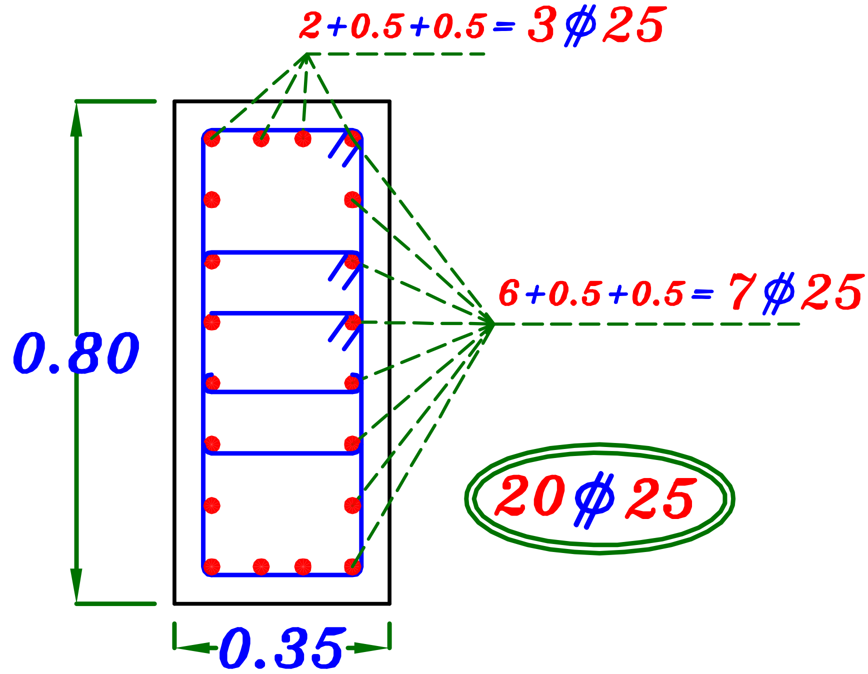
$\therefore A_{sT} > A_{s\,min}$ $\therefore$ Take $A_{sT} = 9600 \ mm^2 = 20\phi25$

نأخذ ٤ أسياخ فى الاركان و ال ١٦ سيخ المتبقيه سيتم توزيعهم على اتجاهى M_X و M_Y

على التوالى بنفس نسبة $A_{sX} : A_{sY}$ أى بنسبة $2604 : 1596$

$$\text{No. of Bars to resist } M_X = \frac{960}{960+3840} * 16 = 3.2 = 4.0 \ \text{bars}$$

$$\text{No. of Bars to resist } M_Y = \frac{4416}{2604+3840} * 16 = 12.8 = 12.0 \ \text{bars}$$



Special Case.

اذا كانت الكمره يؤثر عليها M_Y ، M_X و لا يؤثر عليها P أى $R_b = Zero$

فمن المسموح أن نأخذ قيمه $\alpha_b = 1.0$

أى نصمم على قيم M_Y ، M_X كما هم .

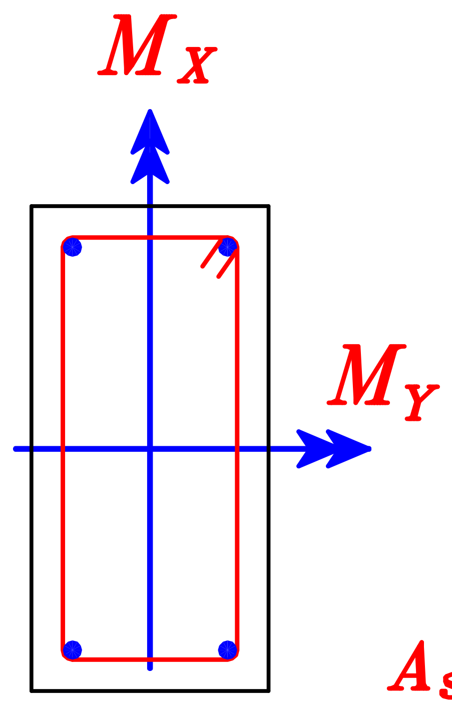
و يتم تصميم قطاع الكمره مرتين :

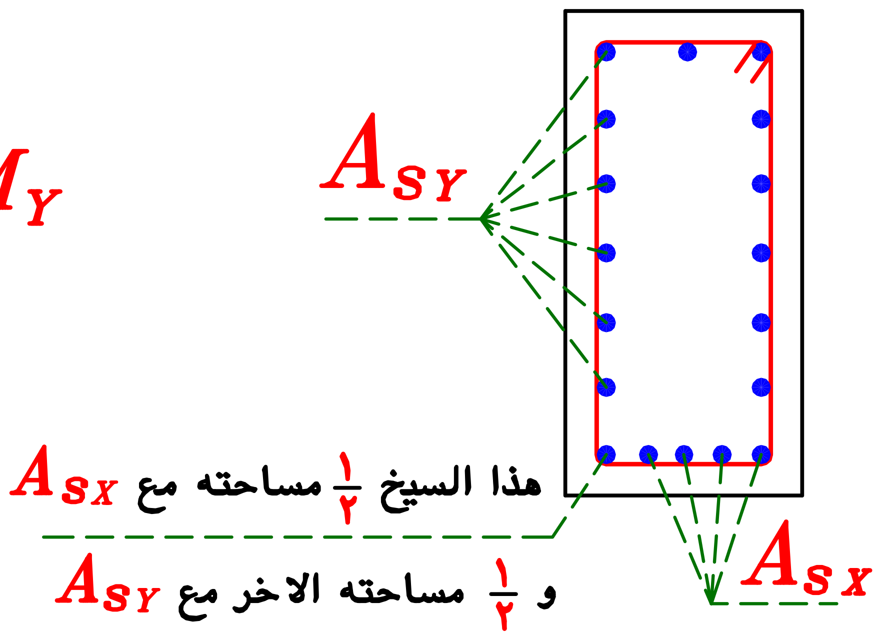
١ـ يتم تصميم قطاع الكمره على M_X فقط و تحديد قيمه A_{sX}

$$Check \quad A_{sX} > A_{smin} = \mu_{min}\, b\, d \xrightarrow{IF\ not} Take\ A_{sX} = A_{smin}$$

٢ـ يتم تصميم قطاع الكمره على M_Y فقط و تحديد قيمه A_{sY}

$$Check \quad A_{sY} > A_{smin} = \mu_{min}\, b\, d \xrightarrow{IF\ not} Take\ A_{sY} = A_{smin}$$





Stiffness.

كلما زادت الـ Stiffness للـ member سواء كان كمره أو عمود كلما كان الـ member أقوى و بالتالى يقل الـ deflection و بالتالى يحمل هذا الـ member جزء أكبر من الحمل .

و نعبر عن الـ Stiffness برمز K
و K تعبر عن نسبه الـ Stiffness
و ليست عن القيمه الفعلية للـ Stiffness

$$K = \frac{E\,I}{L}$$

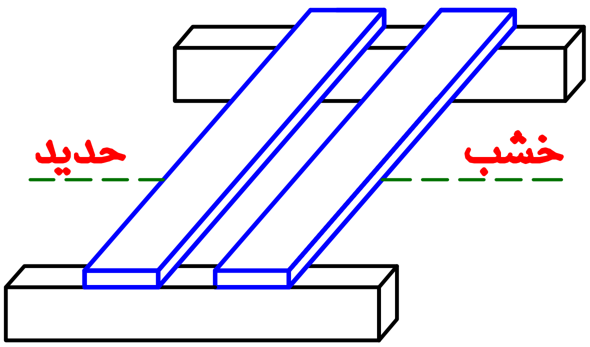
حيث E هى معاير المرونه و يعتمد على الماده المكون منها الـ member
و كلما كانت الماده أقوى كلما زادت قيمه الـ E كلما زادت الـ Stiffness

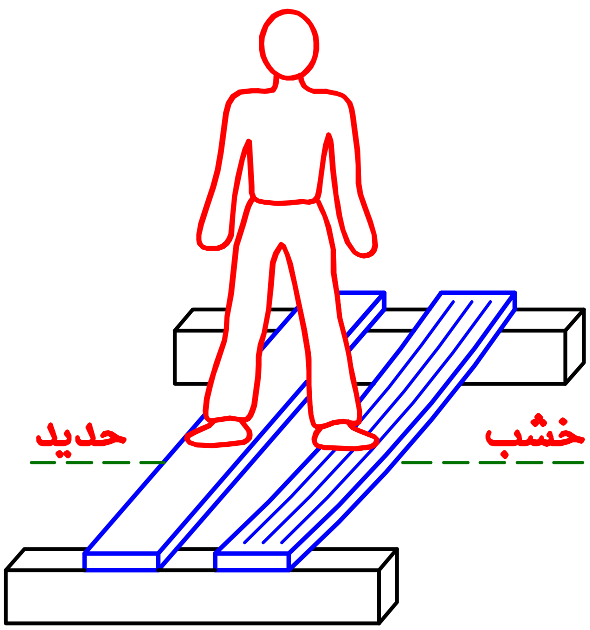
Example.

اذا وجد لوحان لهم نفس القطاع و نفس الطول و لكن نوع الماده مختلف مثلا خشب و حديد اذا وضع حمل على اللوحان سيحدث للخشب deflection أكبر و بالتالى سيحمل الحديد الجزء الاكبر من الحمل .

$$\because\ E_{Steel} > E_{Wood}$$

$$\therefore\ K_{Steel} > K_{Wood}$$





حيث I هى *moment of Inertia* لقطاع ال *member* و يعتمد على

أبعاد القطاع و خاصه ال t $\boxed{I = \frac{b t^3}{12}}$

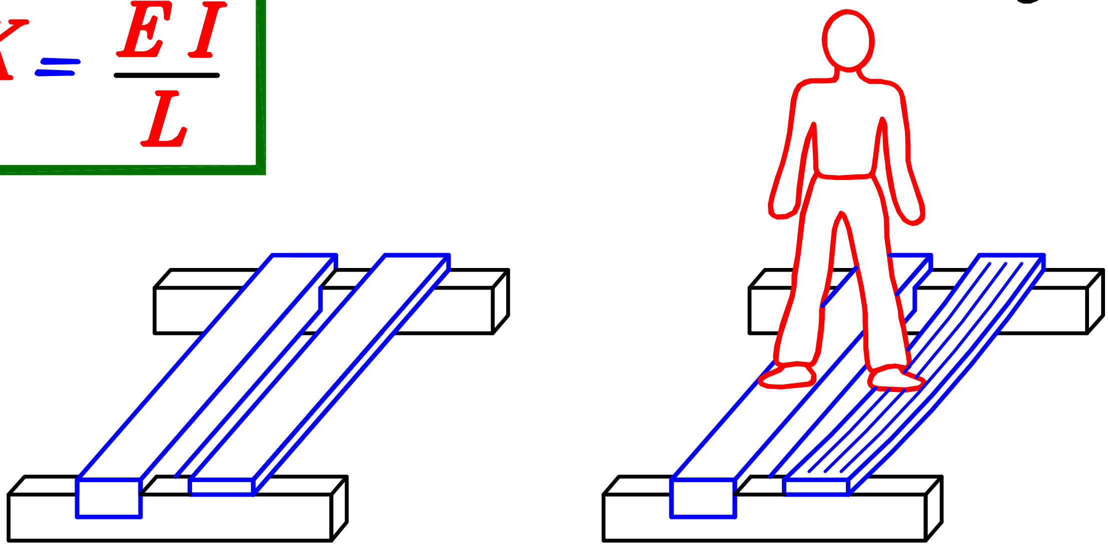
و كلما زادت أبعاد القطاع زادت ال I كلما زادت ال *Stiffness*

Example. اذا وجد لوحان من نفس الماده و لهما نفس الطول و لكن أبعاد قطاعيهما مختلفان

اذا وضع حمل على اللوحان سيحدث للوح الذى له أبعاد أقل *deflection* أكبر

و بالتالى سيحمل اللوح الذى له أبعاد أكبر الجزء الاكبر من الحمل .

$$\boxed{K = \frac{E\,I}{L}}$$



حيث L هى طول ال *member*

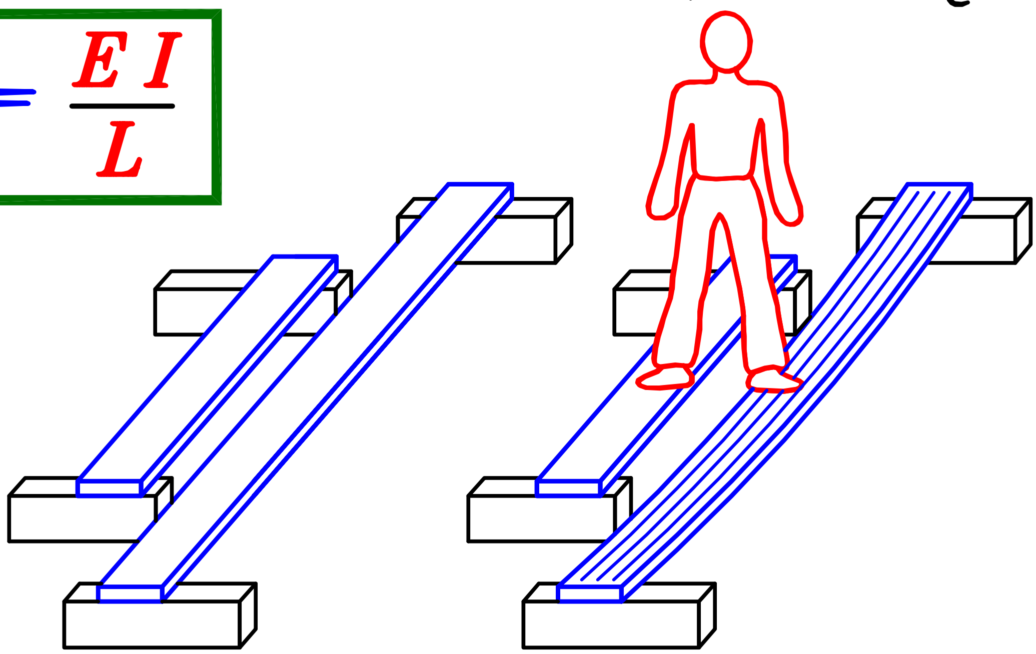
و كلما زاد طول ال *member* L كلما قلت ال *Stiffness*

Example. اذا وجد لوحان من نفس الماده و لهما نفس القطاع و لكن طولهما مختلف .

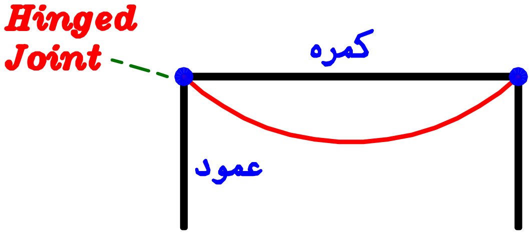
اذا وضع حمل على اللوحان سيحدث للوح الاطول *deflection* أكبر

و بالتالى سيحمل اللوح الاقصر الجزء الاكبر من الحمل .

$$\boxed{K = \frac{E\,I}{L}}$$

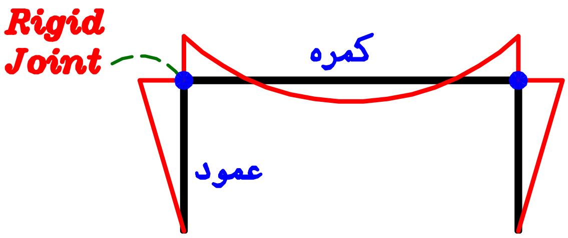


Connection between Beam & Column.



يتكون الـ **B.M.** من الاحمال الموجوده على الكمره

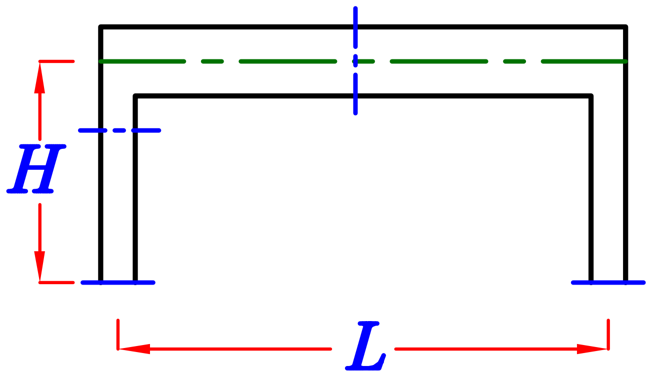
أي أنه دائما الكمره عليها **B.M.**

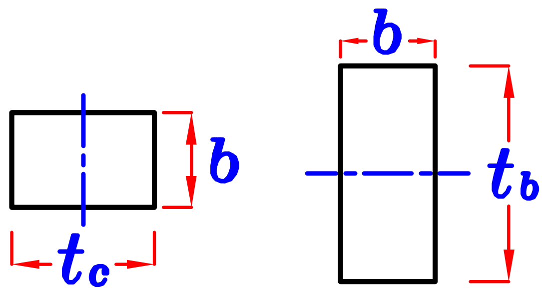


ولكن وجود **B.M.** على العمود يعتمد على الـ **Joint**

بين الكمره و العمود اذا كانت **Hinged** or **Rigid**

و ذلك يعتمد على الـ **Stiffness** بين كلا من الكمره و العمود



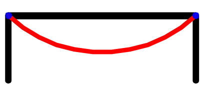


$$I_c = \frac{b\,t_c^3}{12} \qquad I_b = \frac{b\,t_b^3}{12}$$

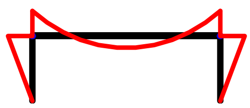
$$\boxed{K_b = \frac{E\,I_b}{L}} \qquad \boxed{K_c = \frac{E\,I_c}{H}}$$

Relative Stiffness.

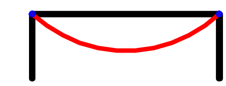
$$\boxed{K_r = \frac{K_b}{K_c}}$$



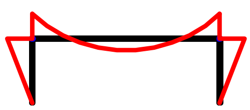
اذا كانت قيمه K_r كبيره نسبيا تعتبر الـ **Joint** بين الكمره و العمود **Hinged**



اذا كانت قيمه K_r صغيره نسبيا تعتبر الـ **Joint** بين الكمره و العمود **Rigid**



للتسهيل سنعتبر أنه عندما تكون $t_c \leqslant \frac{t_b}{2}$ تكون **Hinged Joint**

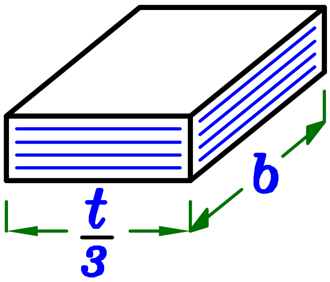


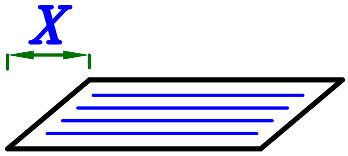
و عندما تكون $t_c \geqslant 0.8\,t_b$ تكون **Rigid Joint**

Real Hinge.

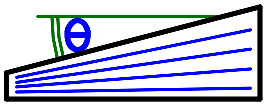
Neoprene Plate.

الـ ***Neoprene Plate*** هى ألواح من الصلب بينها شرائح من المطاط المضغوط.
توضع بين العمود و الكمره أو بين العمود و القاعده لعمل ***Real Hinge***
و فائدتها أنها تسمح بالحركه الافقيه و الدوران .

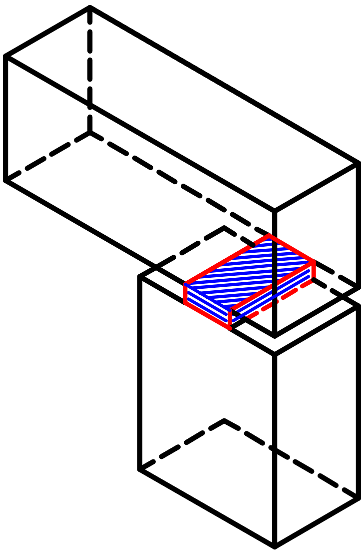




X الحركة الافقية



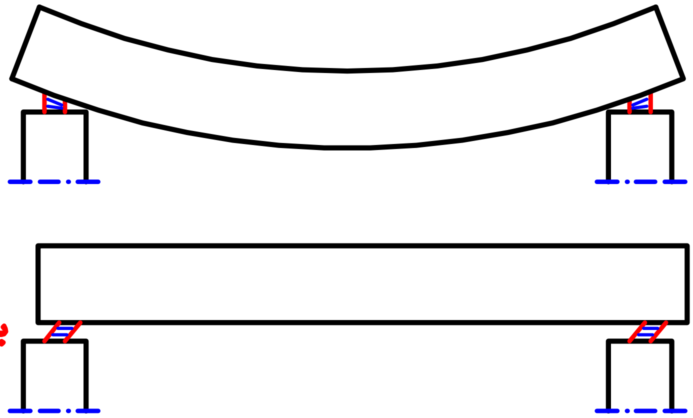
θ الدوران



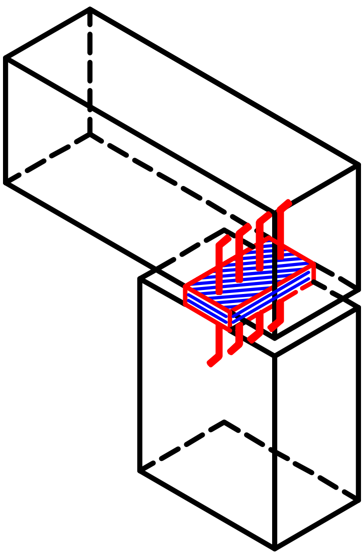
تسمح بالحركه الافقيه و تسمح بالدوران

لذلك تعتبر ***Roller support***

تسمح بالدوران
أى لا تنقل عزوم
على العمود



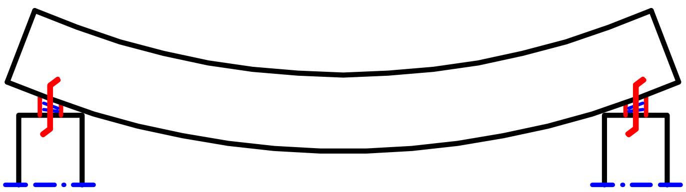
تسمح
بالحركة الافقية



توضع صف أسياخ حديد فى المنتصف تماما
فتمنع الحركه الافقيه و لكن لا تمنع الدوران

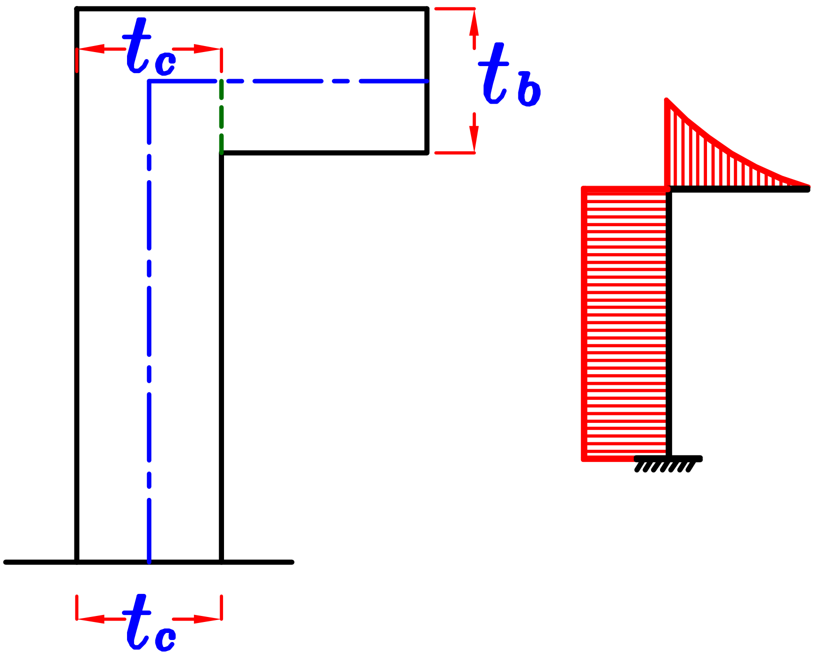
لذلك تعتبر ***Hinged support***

تسمح بالدوران
أى لا تنقل عزوم
على العمود



توجد عده أشكال لاعمده الـ Frames يجب أولا أن نتعلم رسم الخرسانه للاعمده

Concrete Dimensions.



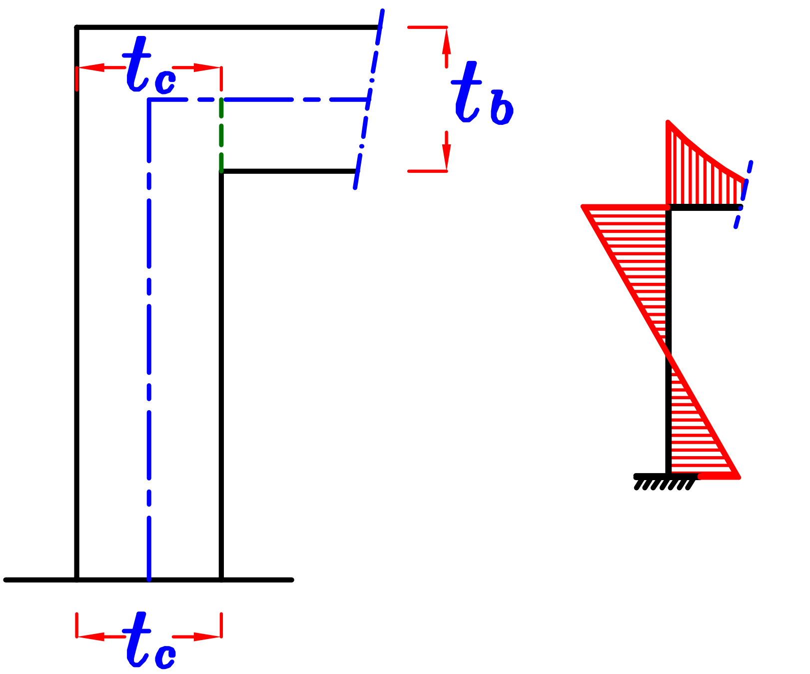
$t_c \geqslant 0.8\, t_b$

حتى تكون Rigid Joint

ملحوظه

ممكن من التصميم

أن تكون $t_c > t_b$



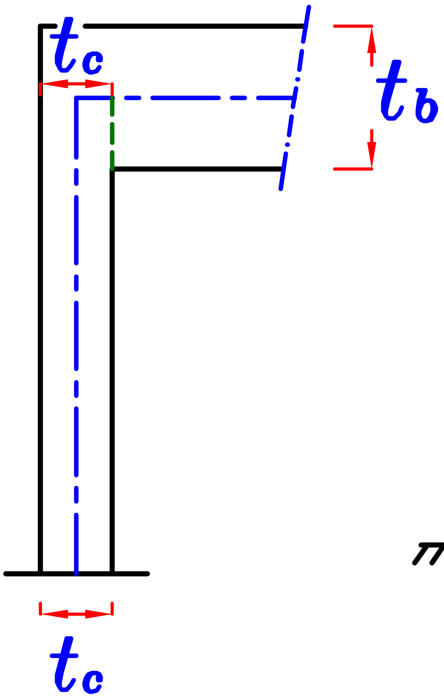
$t_c \geqslant 0.8\, t_b$

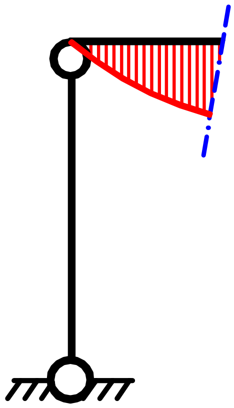
حتى تكون Rigid Joint

ملحوظه

ممكن من التصميم

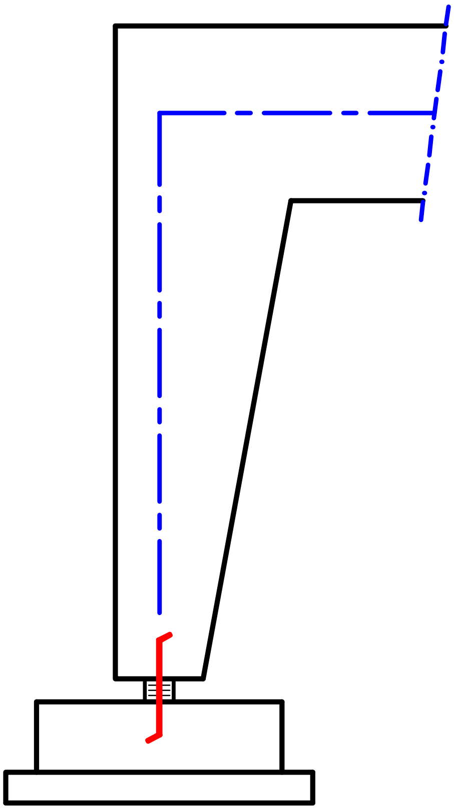
أن تكون $t_c > t_b$

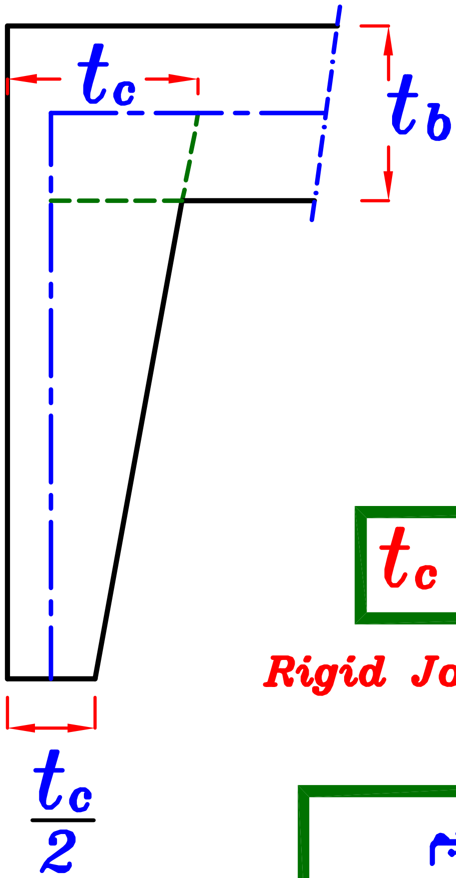


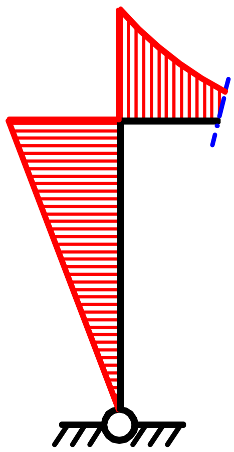


$$\boxed{t_c = \frac{t_b}{2}}$$

حتى تكون Hinged Joint







$$\boxed{t_c \geqslant 0.8\, t_b}$$

حتى تكون Rigid Joint

ملحوظة

ممكن من التصميم

أن تكون $t_c > t_b$

Reinforcement splices.

وصلات التسليح

اذا زاد طول السيخ عن ١٢,- م المفروض أن نعمل وصله فى سيخ الحديد .

Types of splices. أنواع الوصلات

1- *Lap splices.* وصلات بالتراكب

2- *Mechanical splices.* وصلات ميكانيكيه

3- *Welded splices.* وصلات لحام

و الاسهل فى التنفيذ اذا كان ال *member* واقع عليه *compression* or *moment* أن تكون الوصلات بالتراكب .

أما اذا كان ال *member* واقع عليه *tension*
فيجب أن تكون الوصله اما وصله ميكانيكيه او وصله لحام

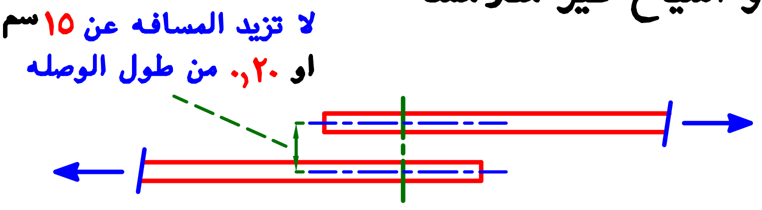
ملحوظه
فى المشاريع الكبرى ممكن عمل أسياخ خاصه للمشروع
طولها أكبر من ١٢,- م و تسمى *special order*
و هى لا تحتاج لوصلات .

1 – *Lap splices.* وصلات بالتراكب

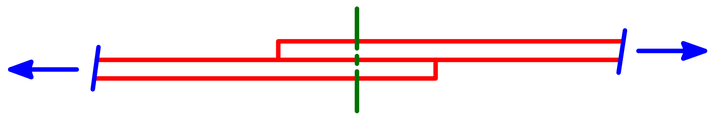
و هى وصلات تعتمد على تراكب الاسياخ

تنتقل الاجهادات بين الاسياخ عن طريق نقلها اولا الى الخرسانه عن طريق قوه التماسك بين الحديد و الخرسانه و أيضا بقوه التماسك تنتقل الاجهادات الى الاسياخ الاخرى .

و تكون وصله التراكب بين اسياخ متلاصمه او اسياخ غير متلاصمه .



أسياخ غير متلاصمه



أسياخ متلاصمه

فى حاله الـ *member* المعرضه لـ *moment*

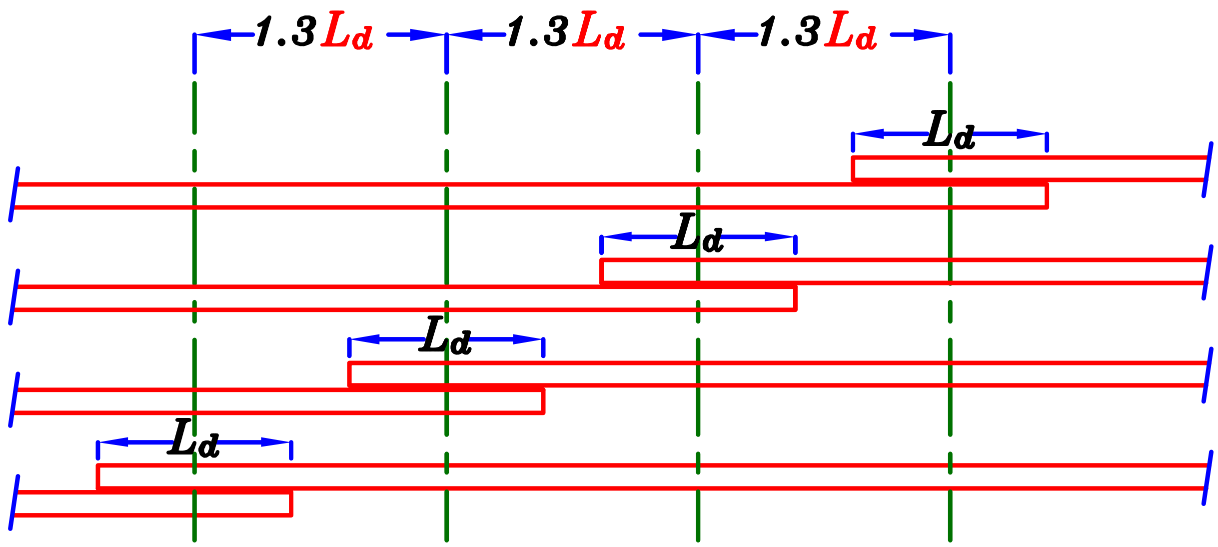
يتم حساب قيمه L_d

$L_d = 60\,\phi * \eta$	$\eta = 1.0$ حديد سفلى $\eta = 1.3$ حديد علوى

يجب ان لا يزيد مساحه الاسياخ الموصوله فى قطاع واحد عن ٢٥ ٪ من المساحه الكليه للاسياخ .

لذا يفضل عمل الوصله للحديد كله على أربع أجزاء .

المسافه من *C.L.* كل وصله و أخرى لا تقل عن $1.3\,L_d$

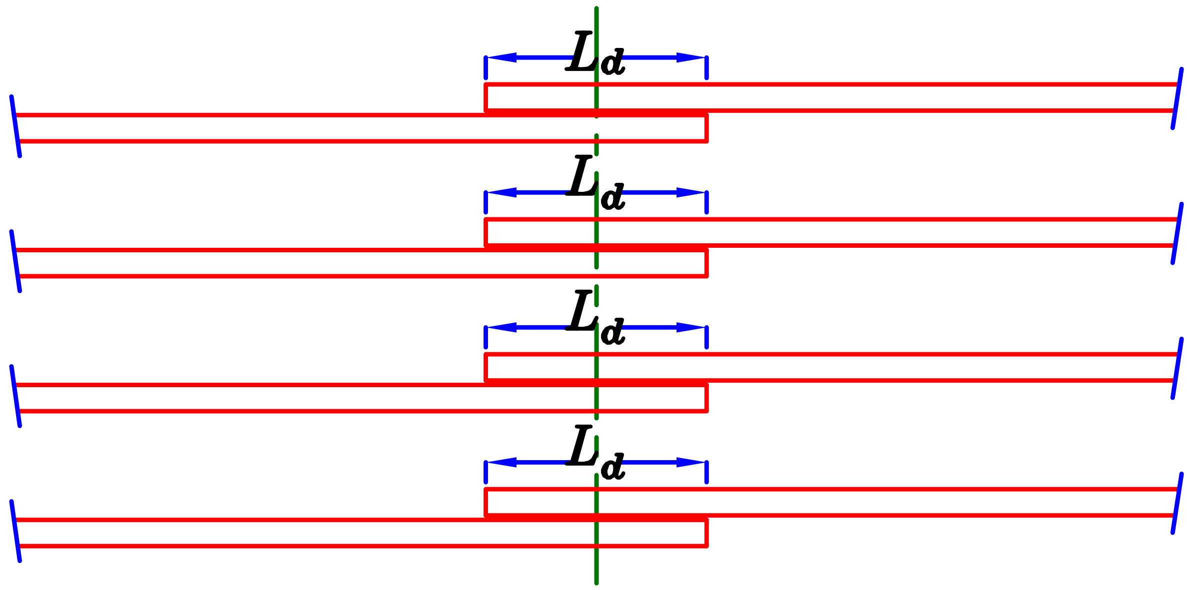


يتم حساب قيمة L_d

$$\boxed{L_d = 40\,\phi * \eta}$$

$\eta = 1.0$ حديد سفلى
$\eta = 1.3$ حديد علوى

ممكن عمل كل الوصلات فى قطاع واحد .



2- Mechanical splices. الوصلات الميكانيكيه

يجب أن لا يقل قطر السيخ عن ١٦ مم $\boxed{min\ \phi\ 16}$

و يستخدم معها جلب من الحديد الصلب مواصفاته لا تقل عن مواصفات الاسياخ الموصوله

كما يجب أن لا تقل مقاومه قطاع الجلبه عن ١,٢٥ مره لـ F_y للاسياخ الموصوله .

و الوصله الميكانيكيه لها طريقتين للتنفيذ :

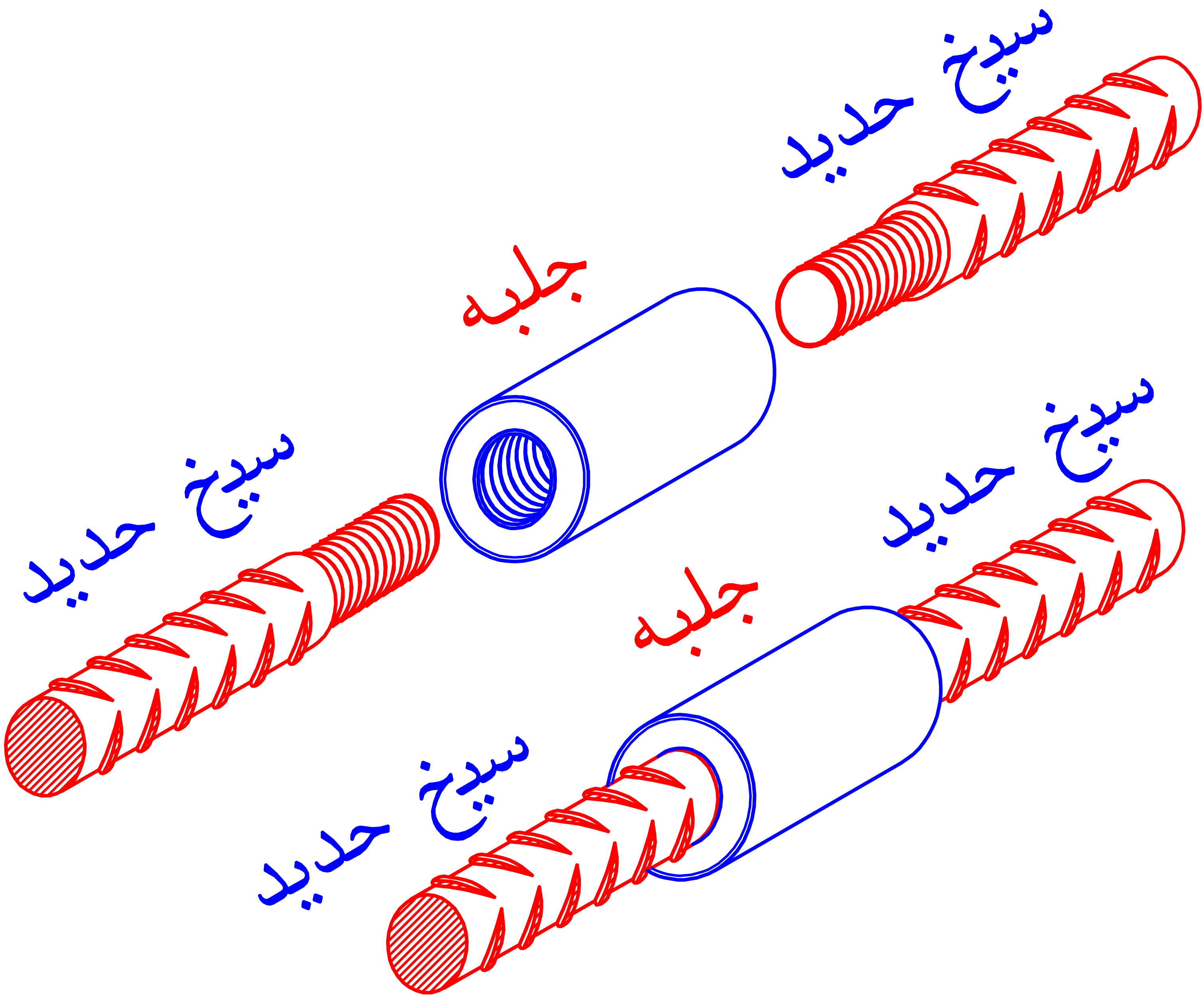
١ـ بقلوظه الاسياخ من الخارج و الجلب من الداخل .

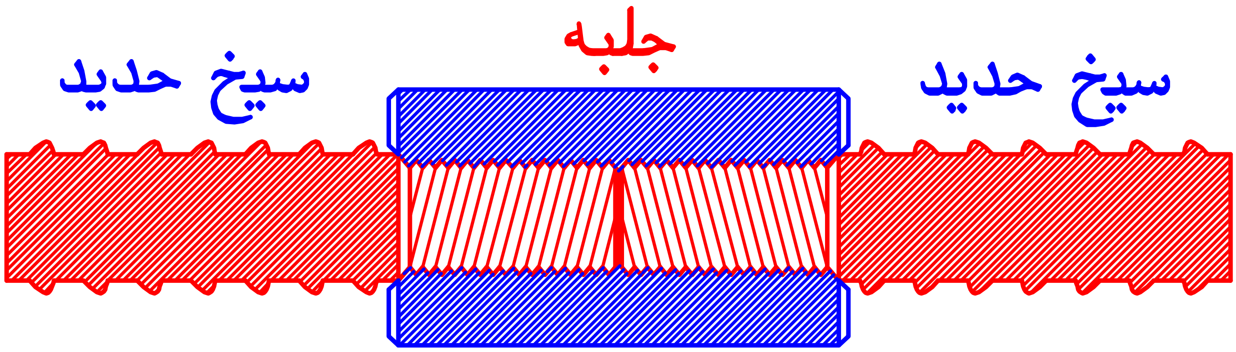
٢ـ بضغط الجلب فى مكابس خاصه على نهايات الاسياخ ذات النتوْات .

و الوصله الميكانيكيه لها طريقتين للتنفيذ :

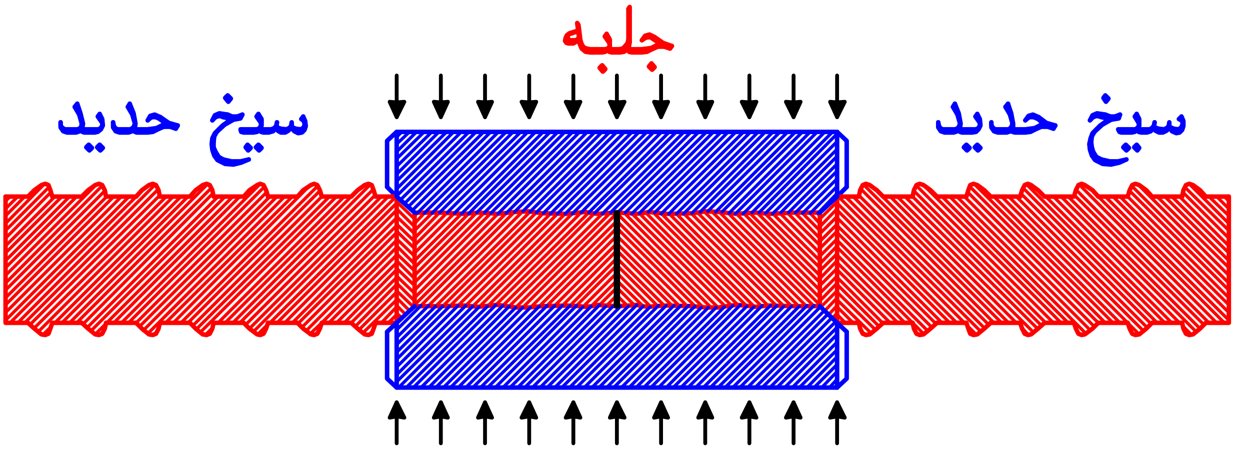
١- بقلوظه الاسياخ من الخارج و الجلب من الداخل .

تنتقل الاجهادات بين الاسياخ بواسطه الارتكاز بين اسنان قلوظ السيخ و اسنان قلوظ الجلبه .





٢- بضغط الجلب فى مكابس خاصه على نهايات الاسياخ ذات النتؤات لتنقل الاجهادات بين الاسياخ بواسطه الاحتكاك بين السطح الداخلى للجلبه مع السطح الخارجى لنهايه الاسياخ .



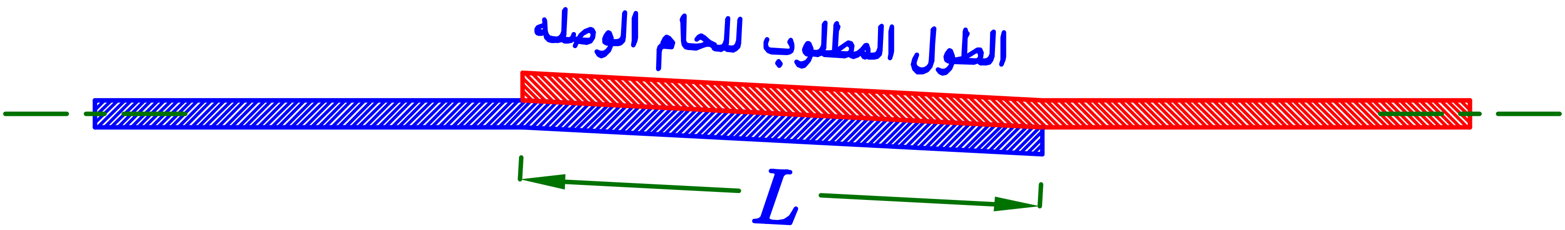
3- Welded splices. وصلات اللحام

يجب أن لا يقل قطر السيخ عن ١٦ مم $\boxed{min \phi 16}$

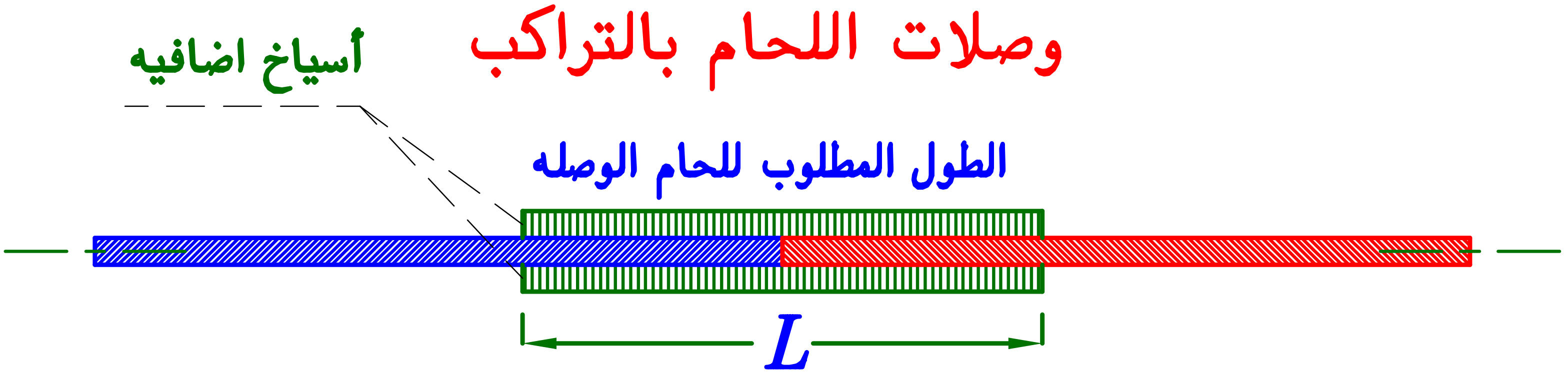
١- يستخدم لحام كهربائى .

٢- يجب أن يكون محور السيخين الملحومين على استقامه واحده .

٣- يجب ان لا تزيد مساحه الاسياخ الملحومه فى قطاع واحد عن ٢٥ ٪ و باقى الوصلات على مسافات طوليه لا تقل عن ٢٠ مره قطر السيخ الملحوم .



وصلات اللحام بالتراكب



وصلات اللحام بأستخدام أسياخ اضافيه

Reinforcement of Frames.

Introduction.

يتكون الـ ***Frame*** من أربع عناصر أساسيه :

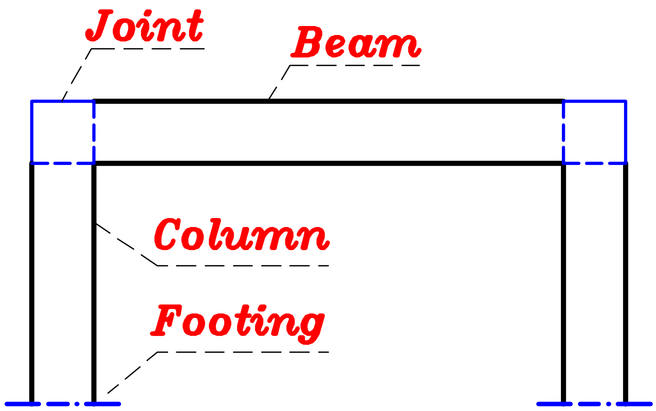
١- الكمرات (أفقيه أو مائله)

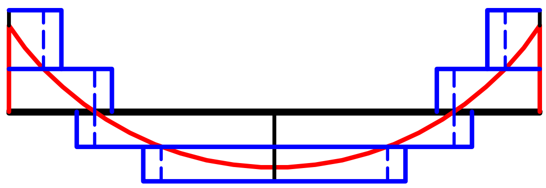
٢- الاعمده (رأسيه أو مائله)

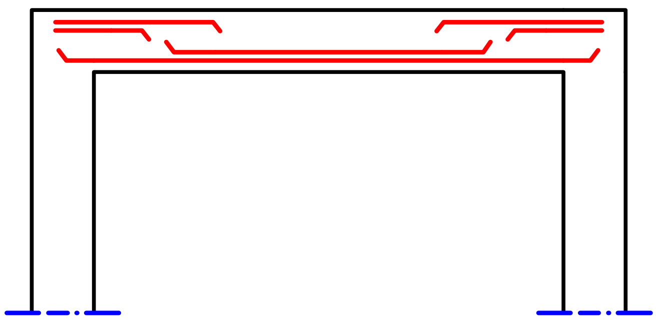
٣- الـ ***Joints*** و هى التى تربط العناصر ببعضها

(***Rigid Joint or Hinged Joint***)

٤- القواعد .

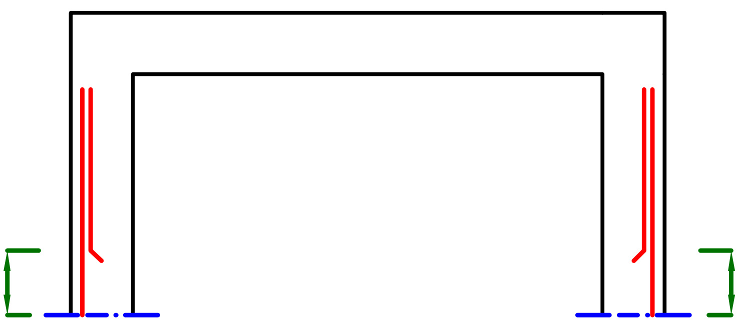






نرسم التسليح فى الكمرات و يقف عاده

بطريقه ***Moment of Resistance*** (M_R)



يفضل رسم التسليح فى الاعمده

بطريقه ***Empirical Method***

لان ***Normal Force*** على الاعمده

كبيره و بالتالى طريقه (M_R)

لن تكون دقيقه .

يتم ربط تسليح الكمرات و الاعمده

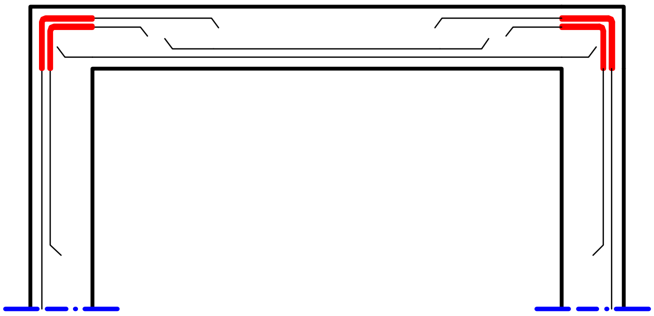
بتسليح ال ***Joints***

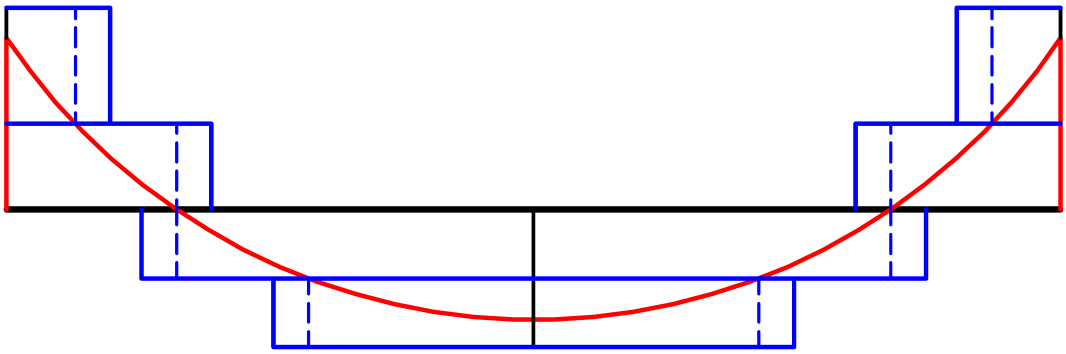
مع ملاحظه انه سيكون هناك حديد مشترك

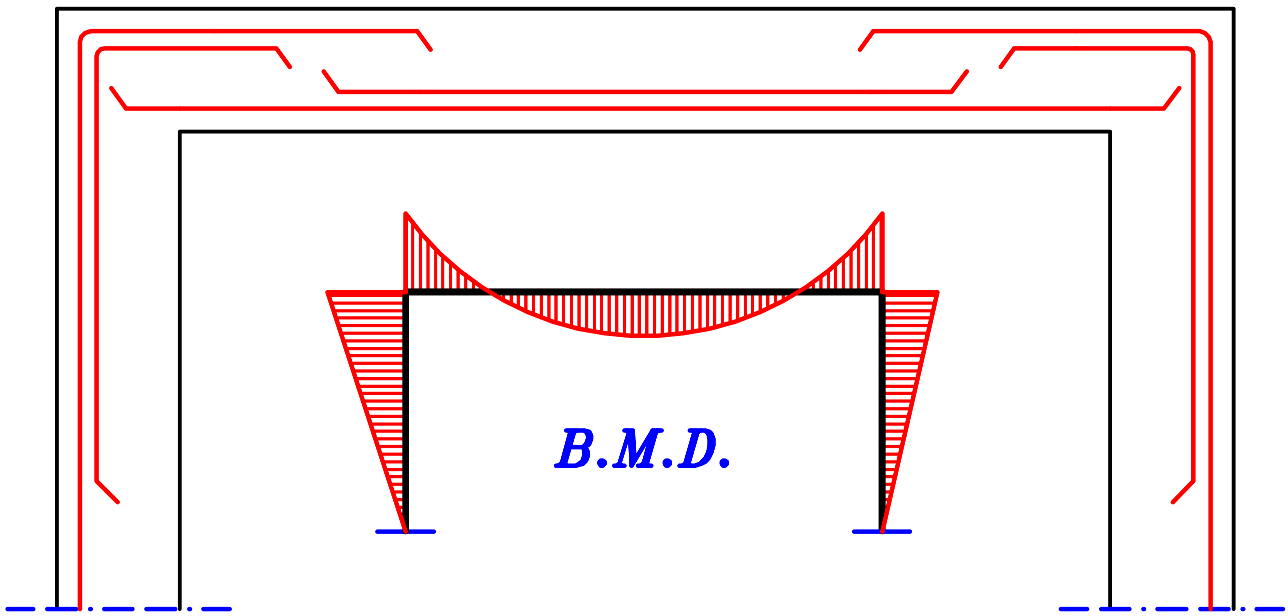
بين الكمرات و الاعمده

مما سيتلزم ان توحد قطر الاسياخ ϕ

فى هذه القطاعات .





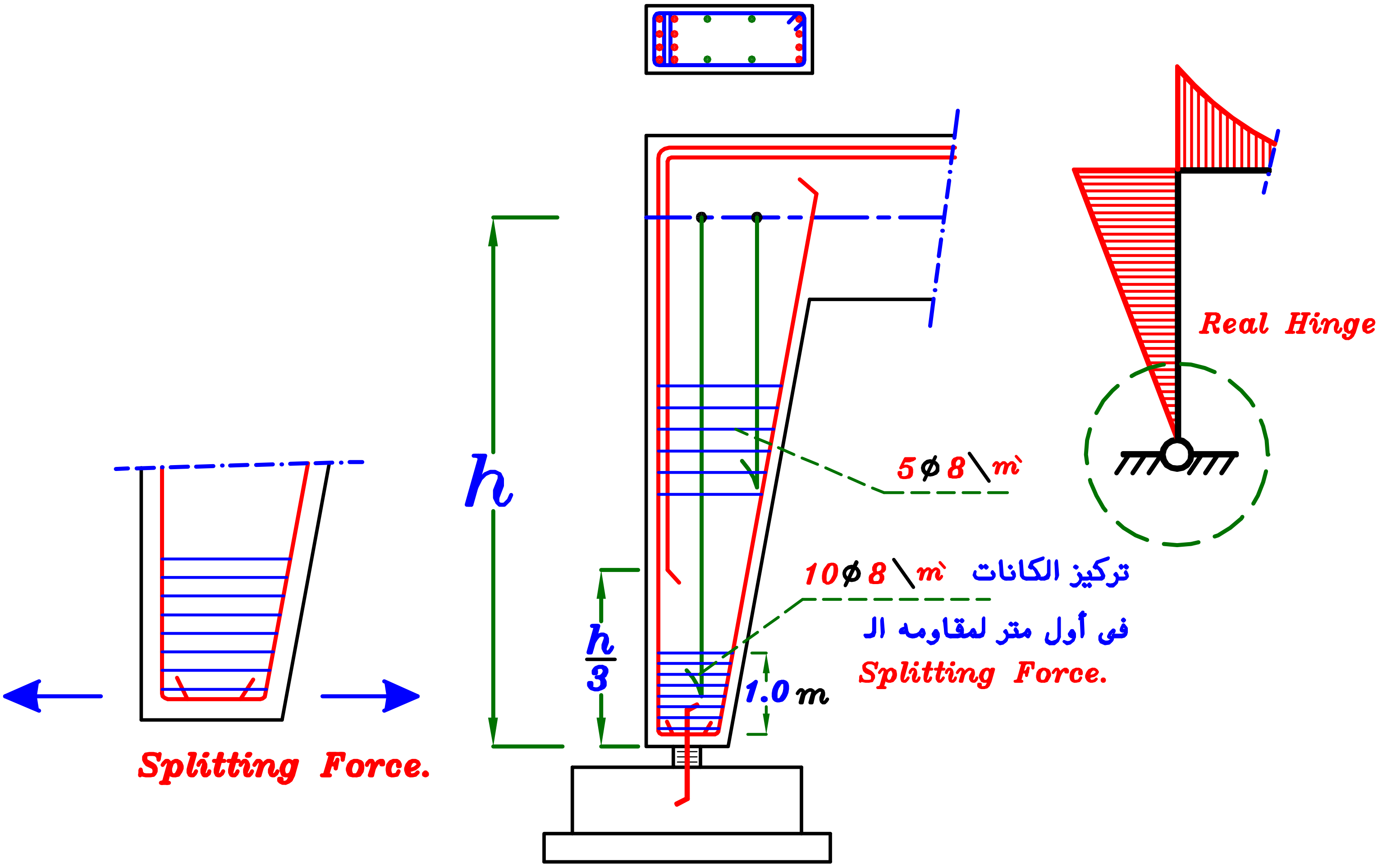


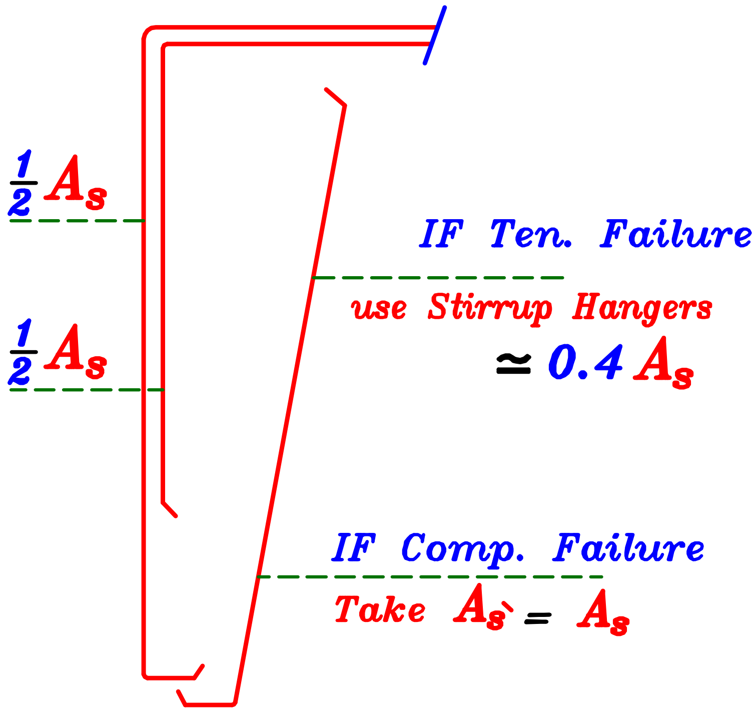
ملحوظه

تسليح القواعد لن ندرسه فى هذا الملف و سيدرس لاحقا فى ملف أخر بأذن الله

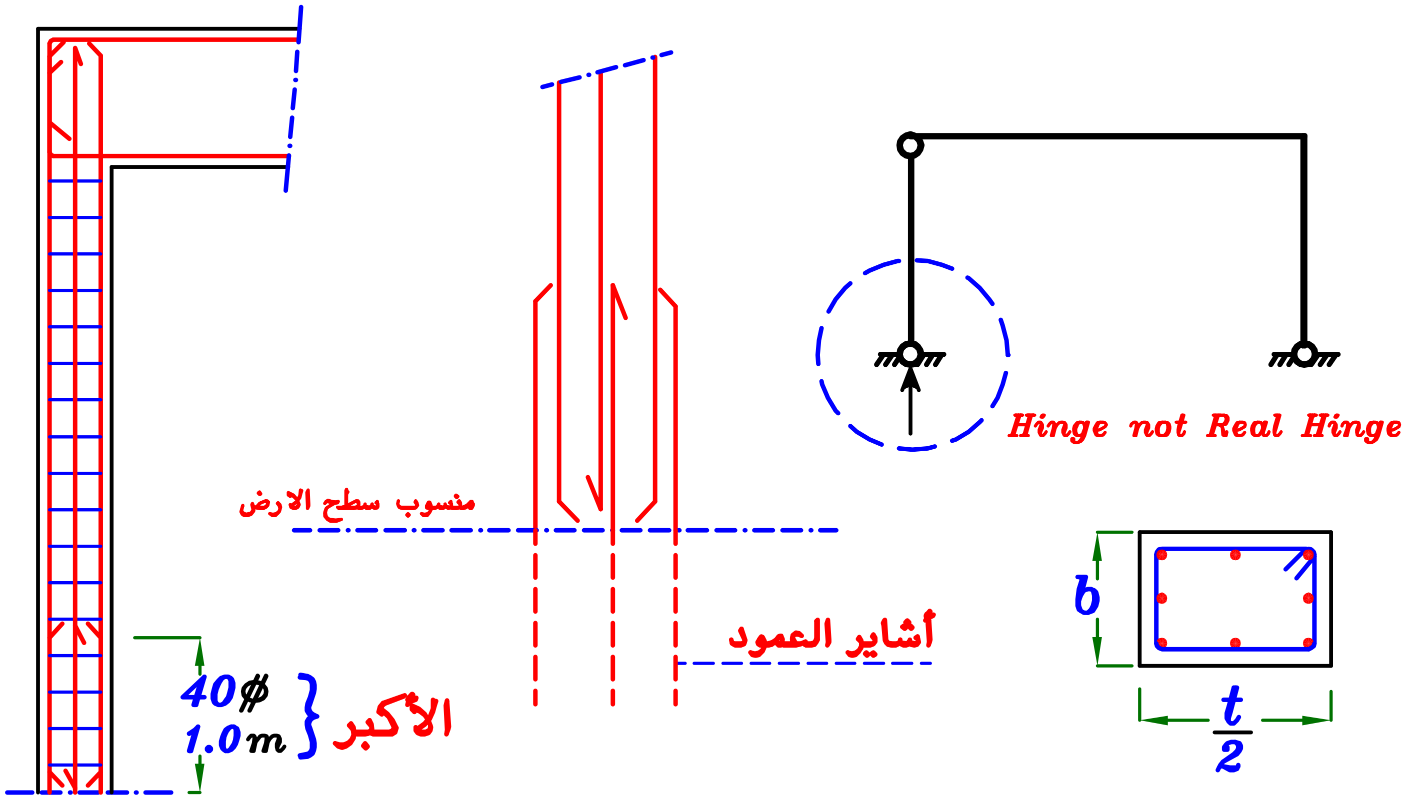
RFT. of columns (Using Empirical Method)

① Hinged–Rigid Column.

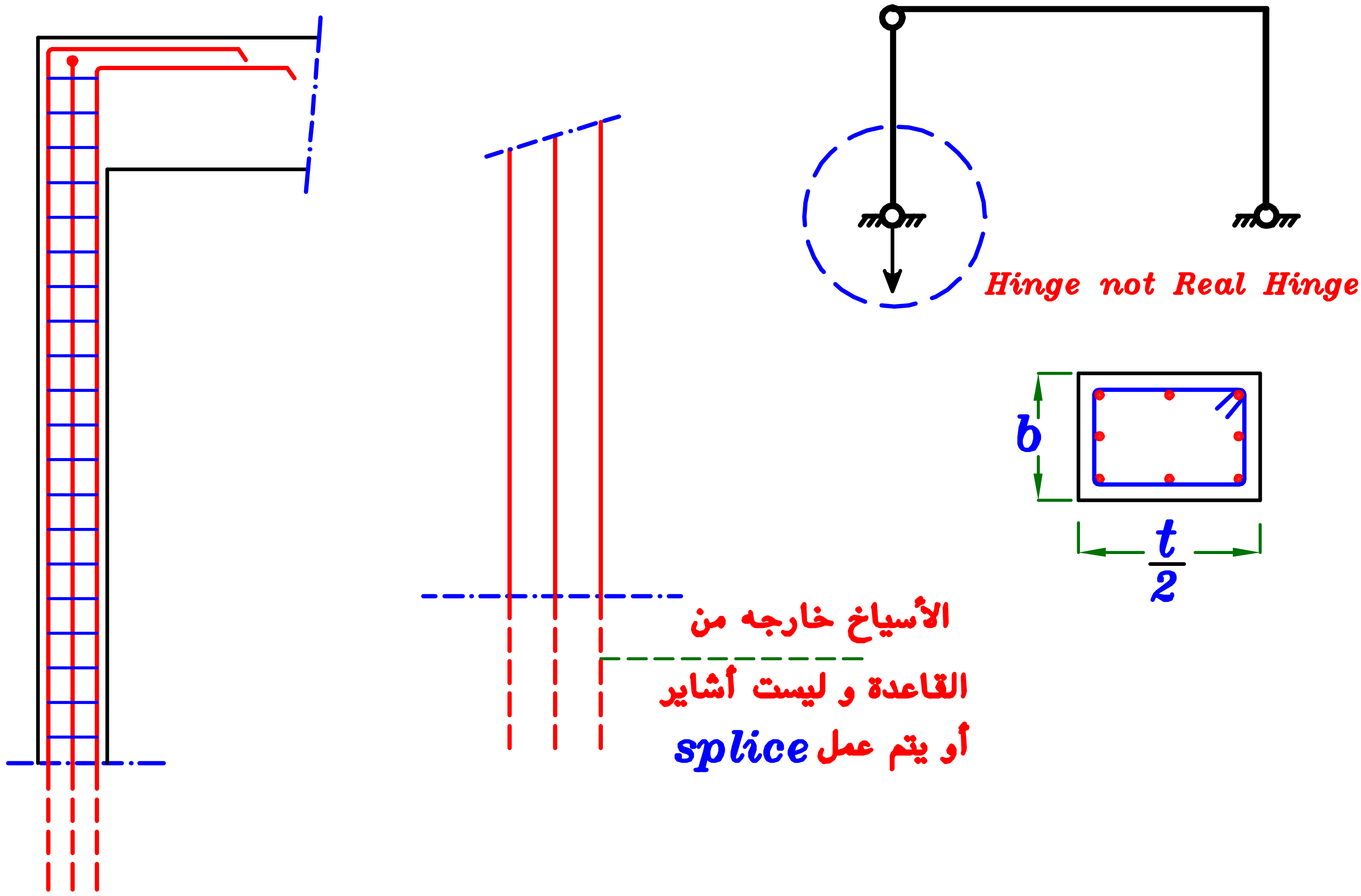




② Link member. (For Compression Link member)

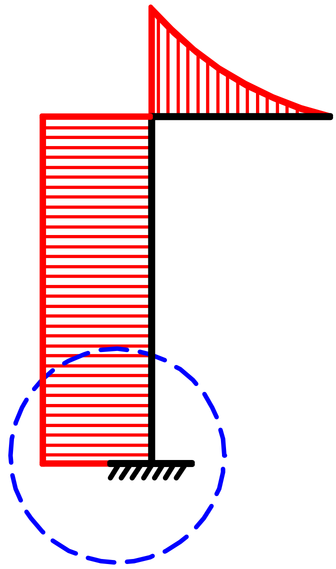


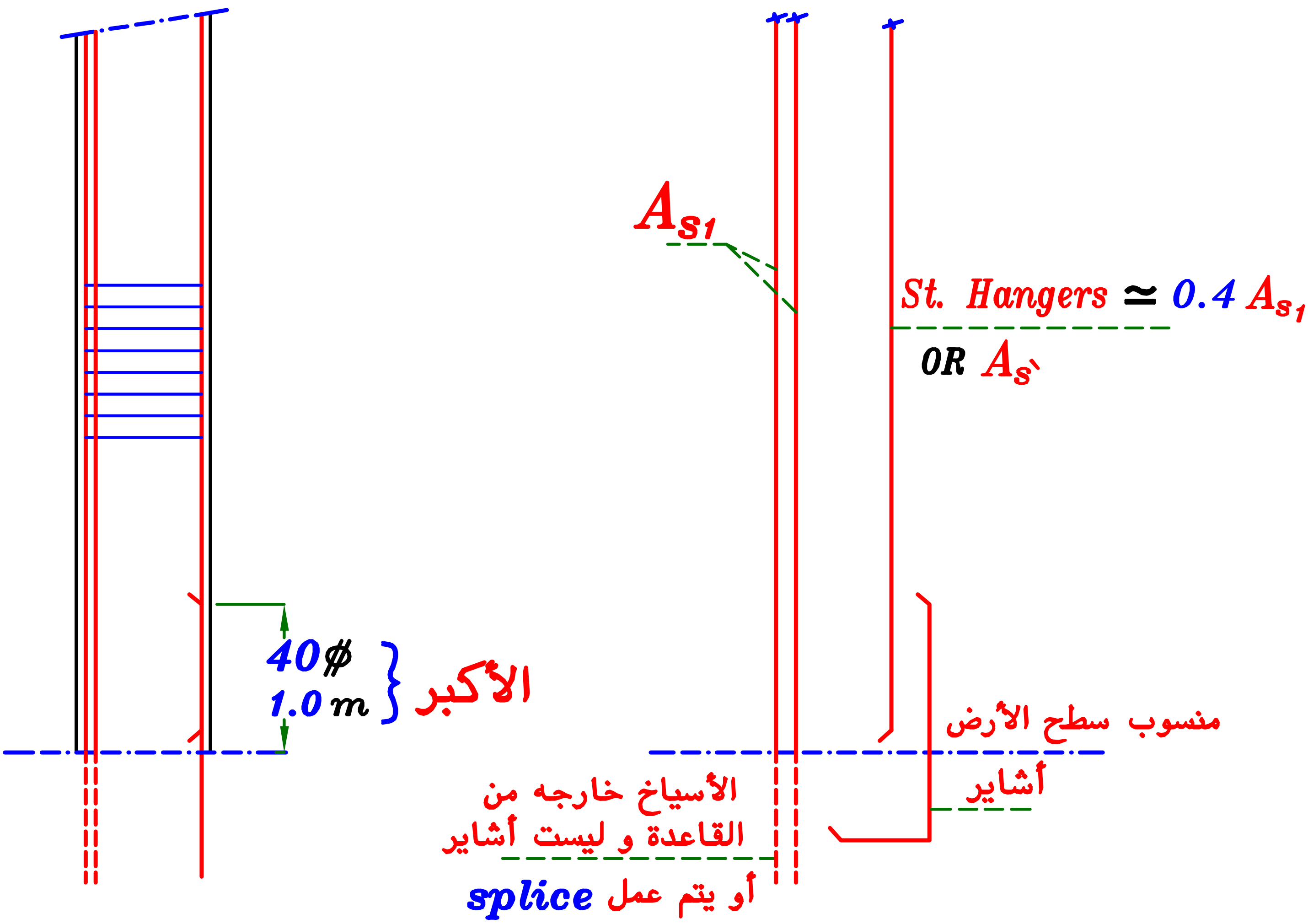
Hinged Support. (For Tension Link member)



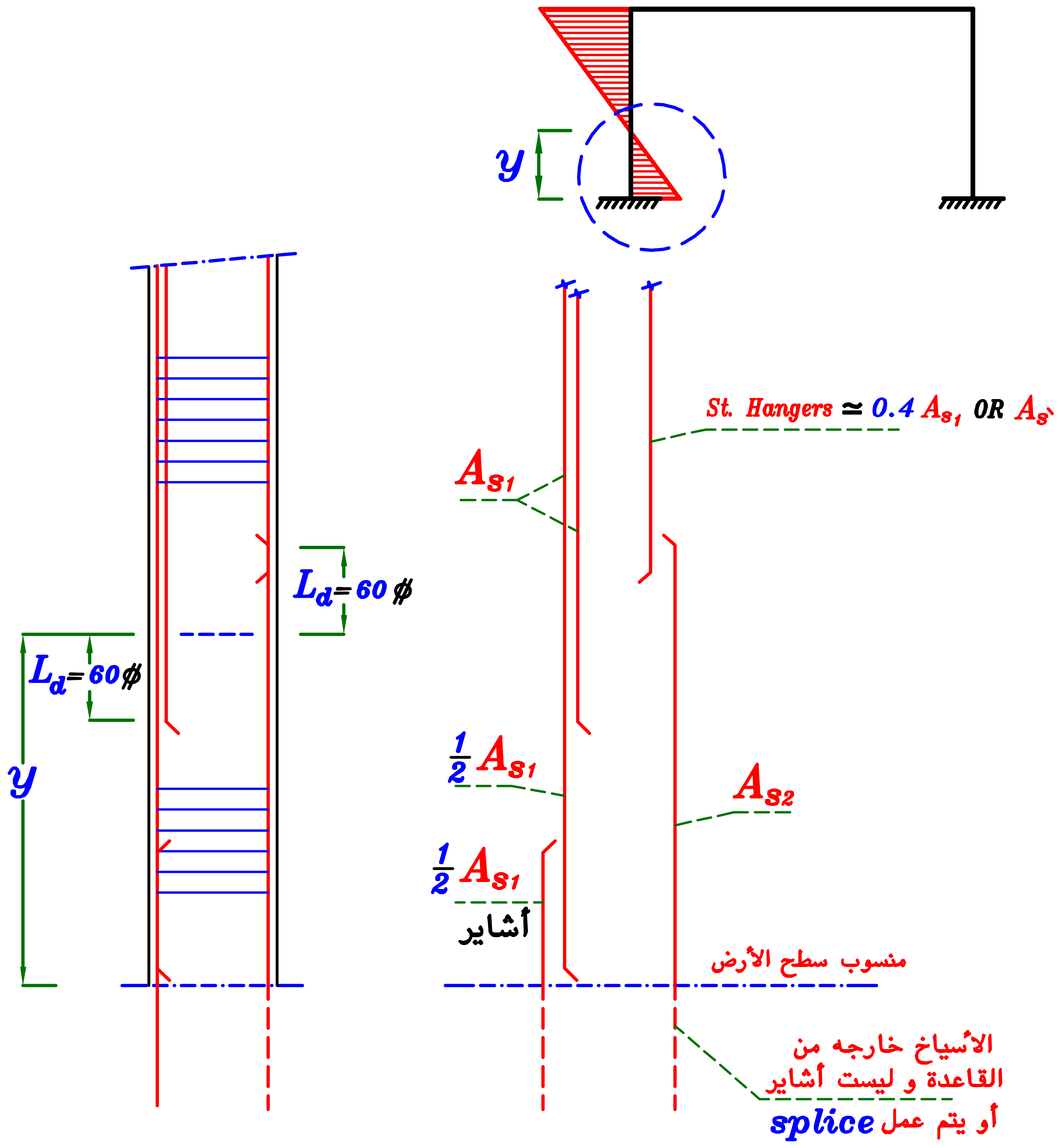
③ Rigid–Rigid Column

ⓐ Moment at one side only.





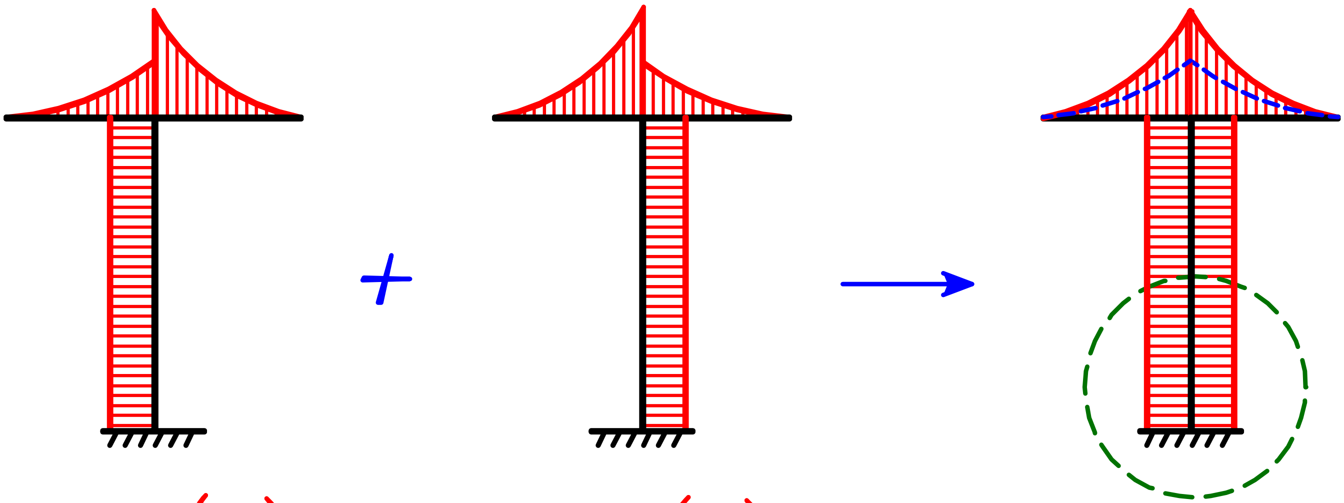
ⓑ *Moment at Two sides.*



ملحوظه

الاشاير تكون جهه الـ *Compression* فقط و تكون بنفس قيمه التسليح الرئيسى
أما جهه الـ *Tension* فيجب أن يخرج التسليح الرئيسى من القاعده
أو يتم عمل *Lap splice*

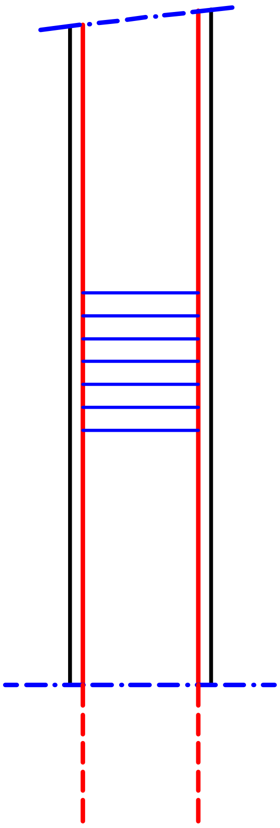
© Moment at Two sides due to Cases of Loading.

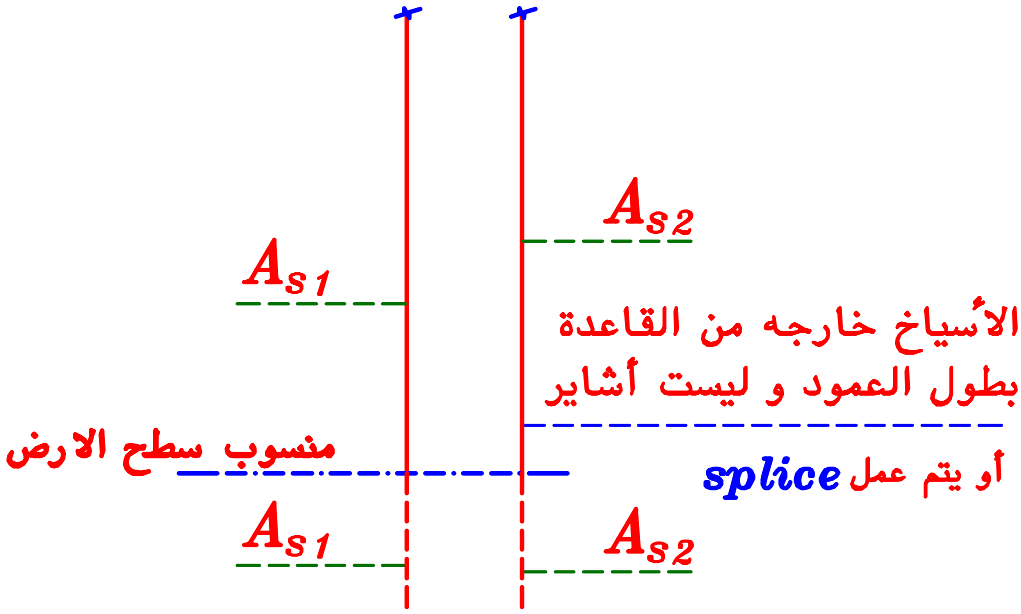


Case (1) Case (2) max–max B.M.D.

Symm. حديد متماثل لأنه

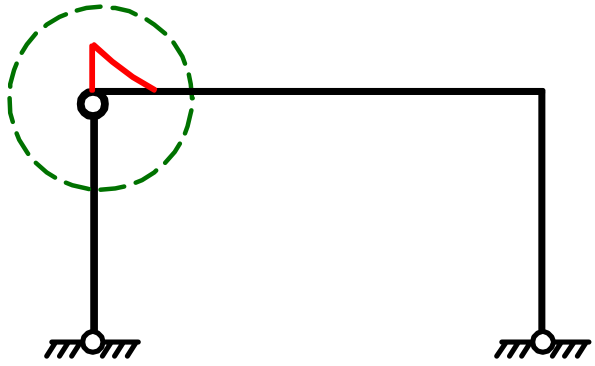
$$A_{s1} = A_{s2}$$

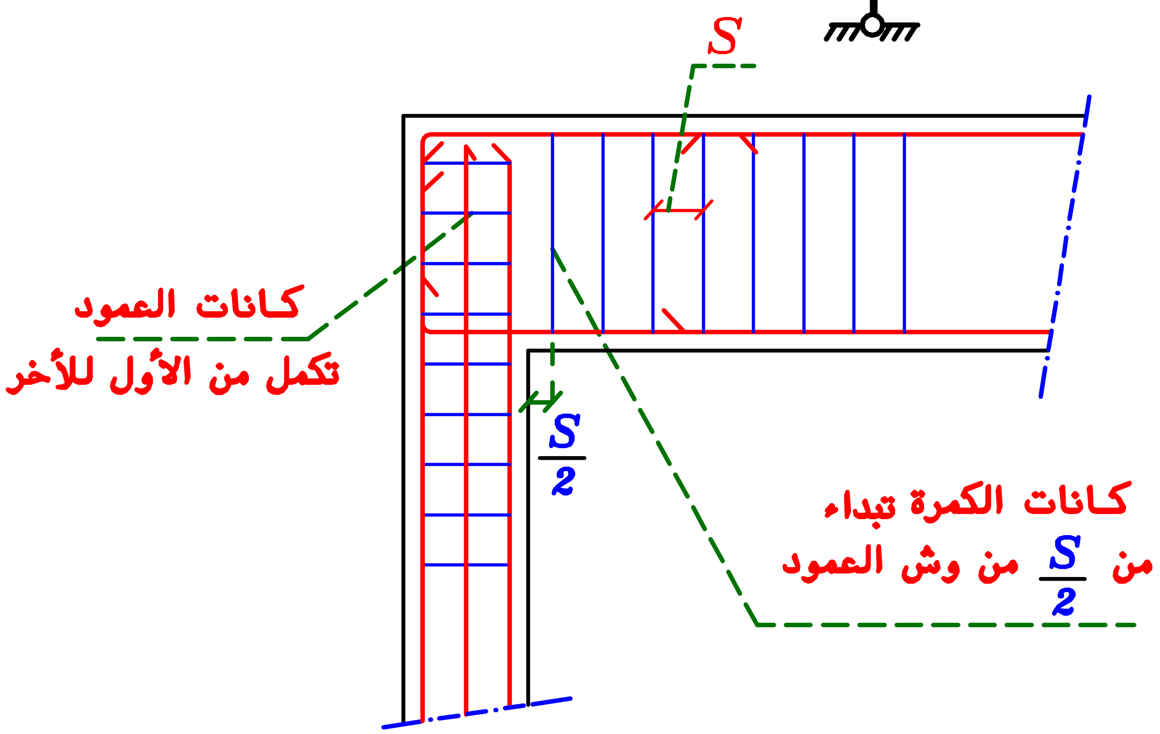


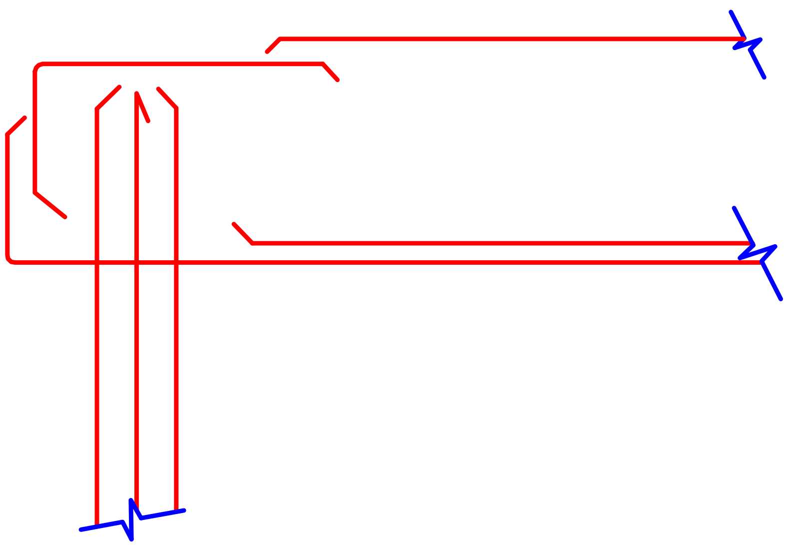


① Hinged Joints. (Joint between the beam & Link member)

هذه ال joint لا تنقل عزوم



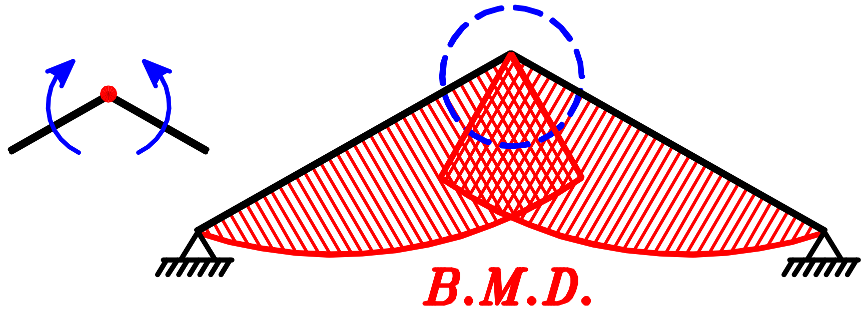




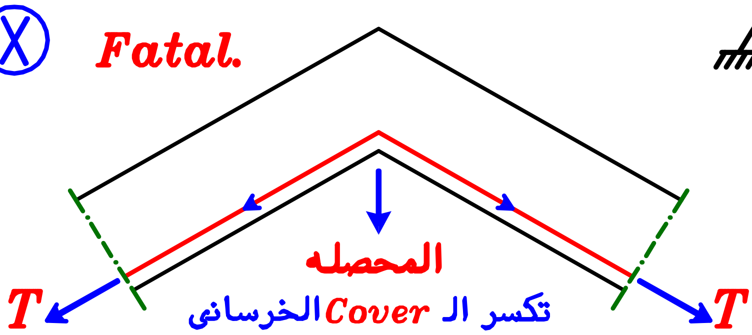
② Rigid Joints. joints تنقل عزوم

ⓐ Opening Joints يحدث عندها تداخل فى العزوم joints

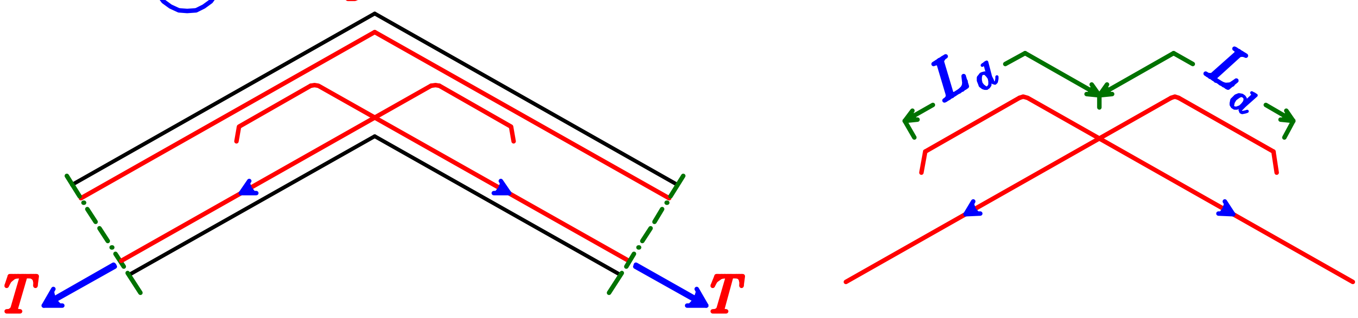
ملحوظه : إذا حدث تداخل فى العزوم يجب عمل مقص أو فيونكه



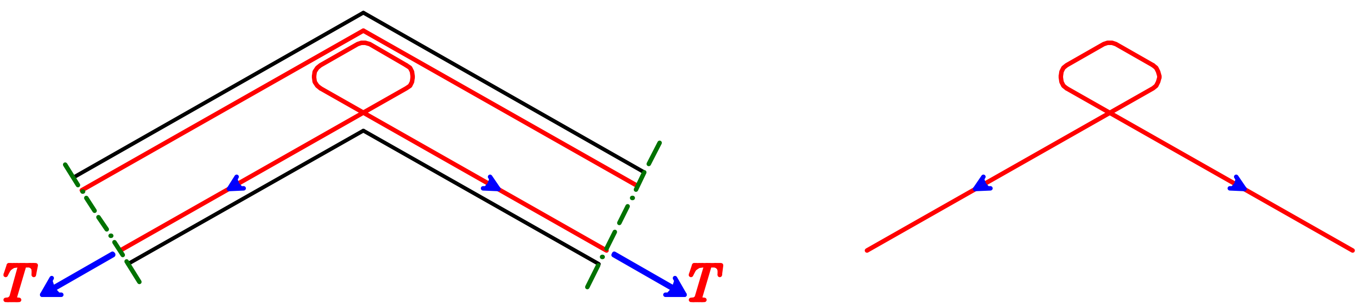
Ⓧ Fatal.



✓ مقص



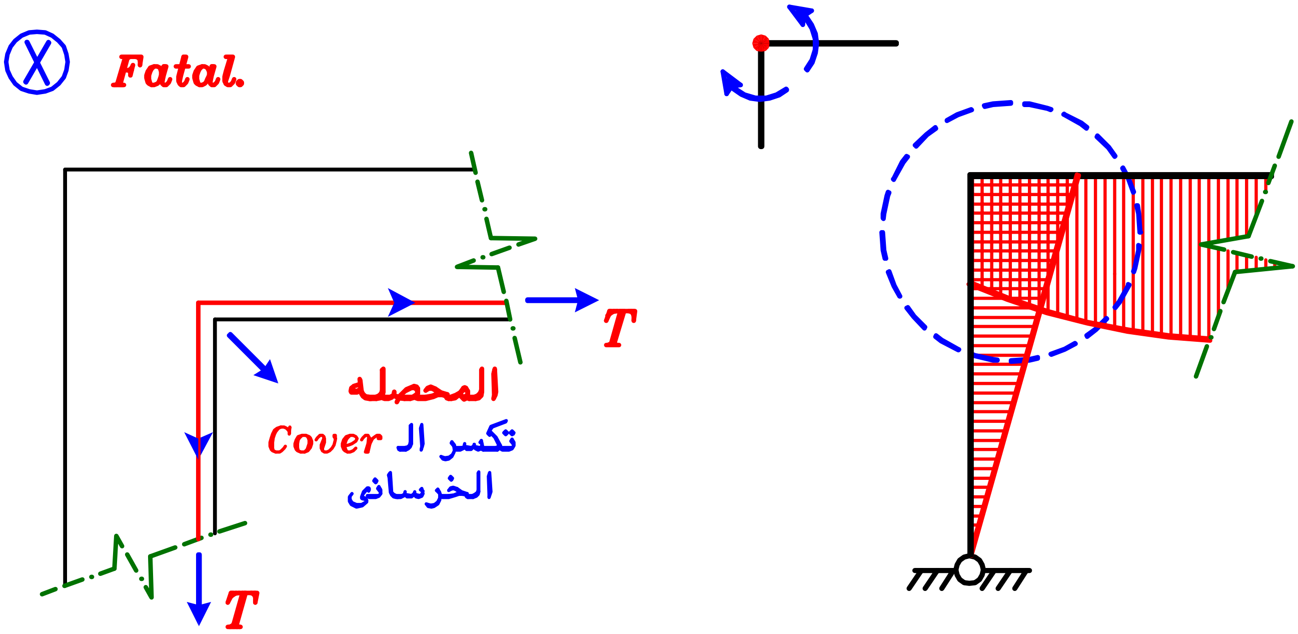
✓ فيونكه (كرفاته)

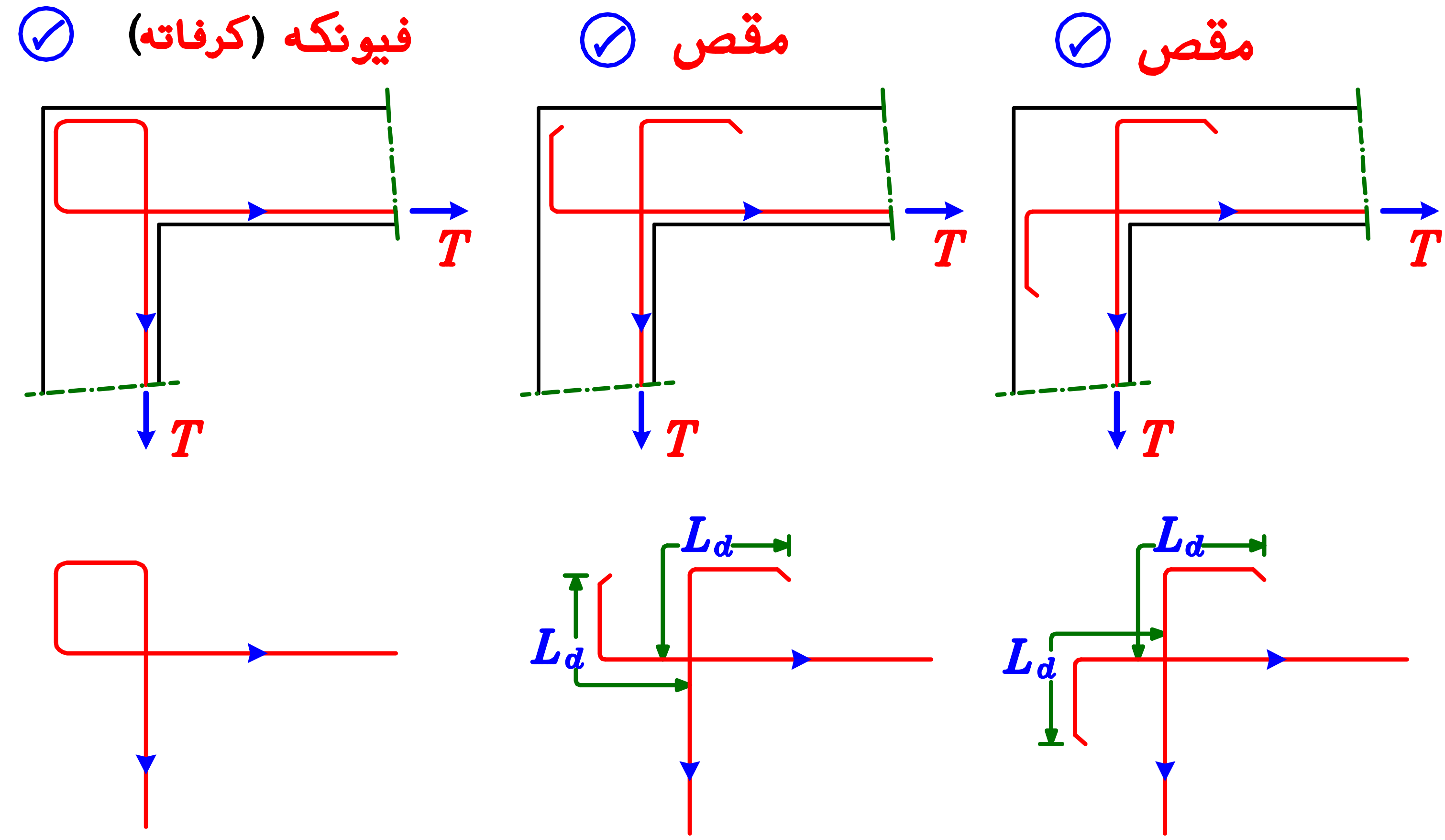


ملحوظه : يفضل عمل المقص عن الفيونكه .

يوجد تداخل فى العزوم

إذاً يجب عمل مقص أو فيونكه

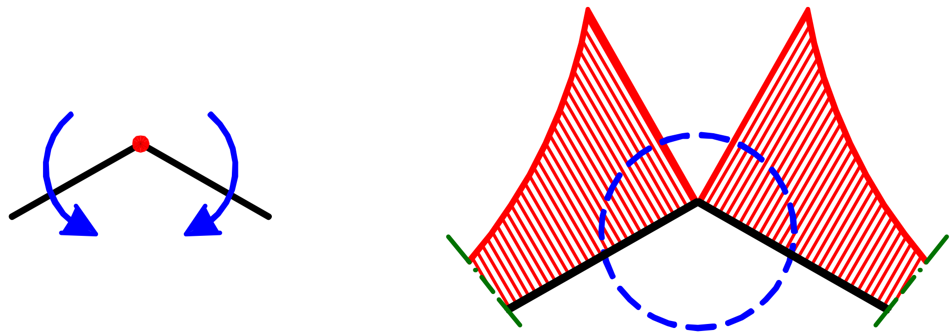




يفضل هذا الحل فى التنفيذ

حيث يتم صب العمود أولا

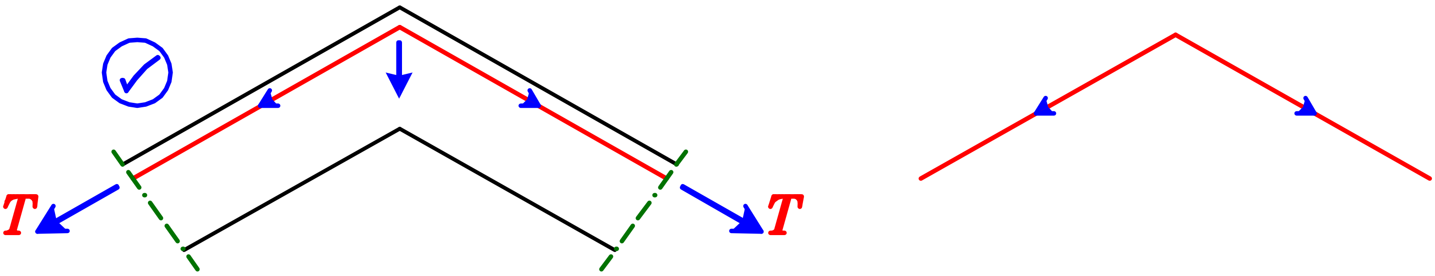
ⓑ *Closing Joints* يحدث عندها تباعد فى العزوم *Joints*



B.M.D.

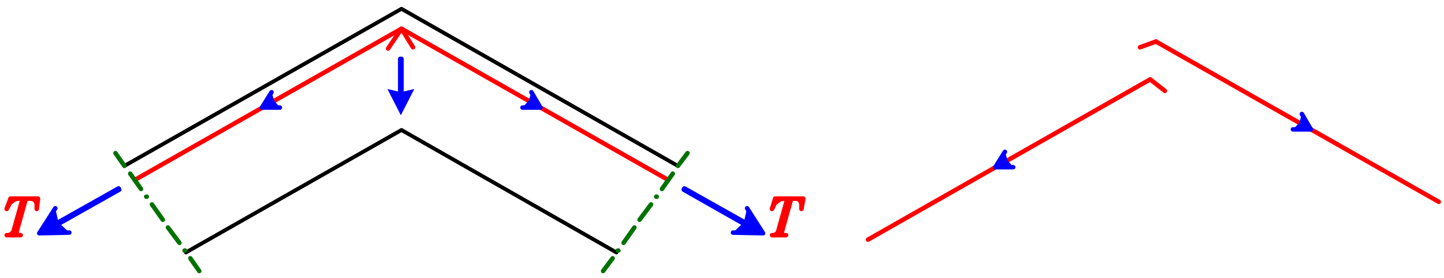
لا يوجد خوف على الخرسانه لأن سمكها كبير

✓



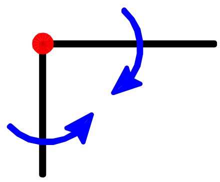
يجب أن يكمل حديد الشد دون أن ينقطع فى منطقة الـ *max. moment*

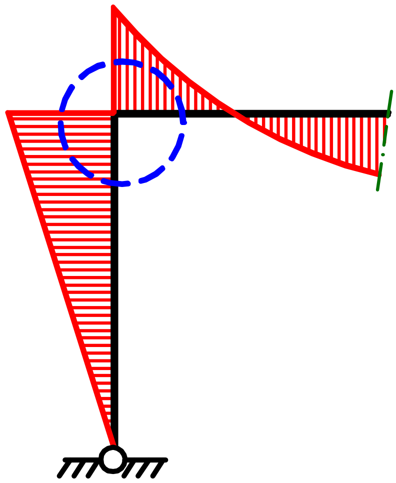
Ⓧ *Fatal.*

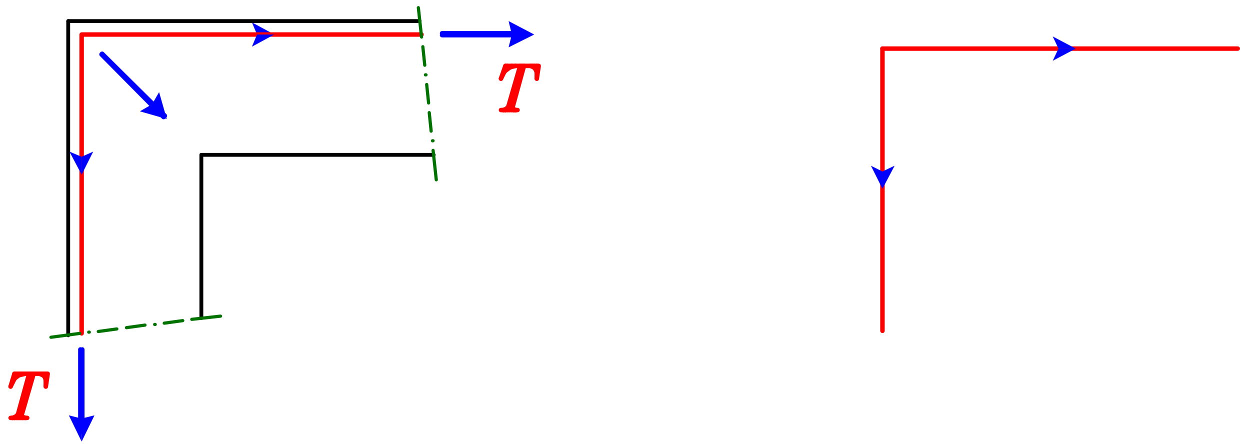


Fatal

توقيف الحديد عند منطقة الـ *max. moment*

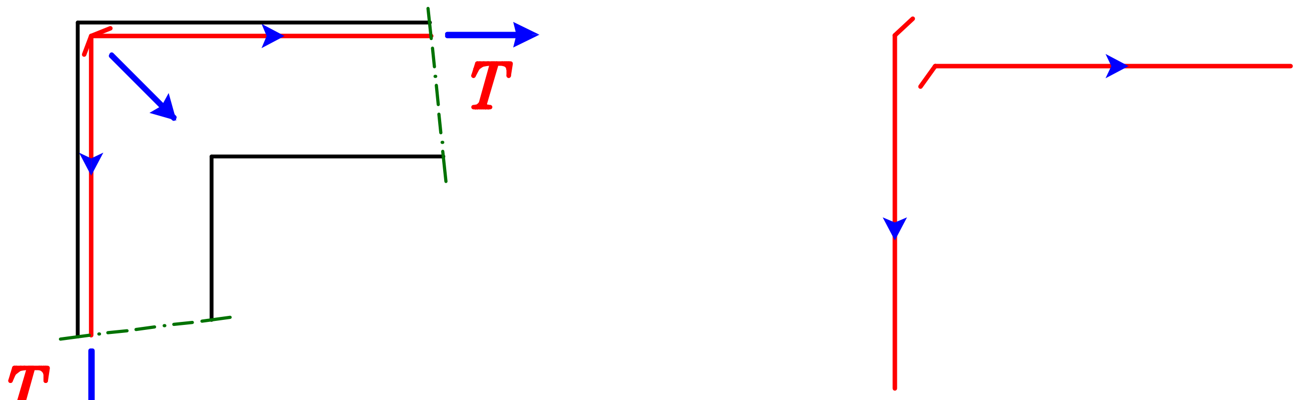






يجب أن يكمل حديد الشد دون أن ينقطع فى منطقة ال ***max. moment***

Fatal.

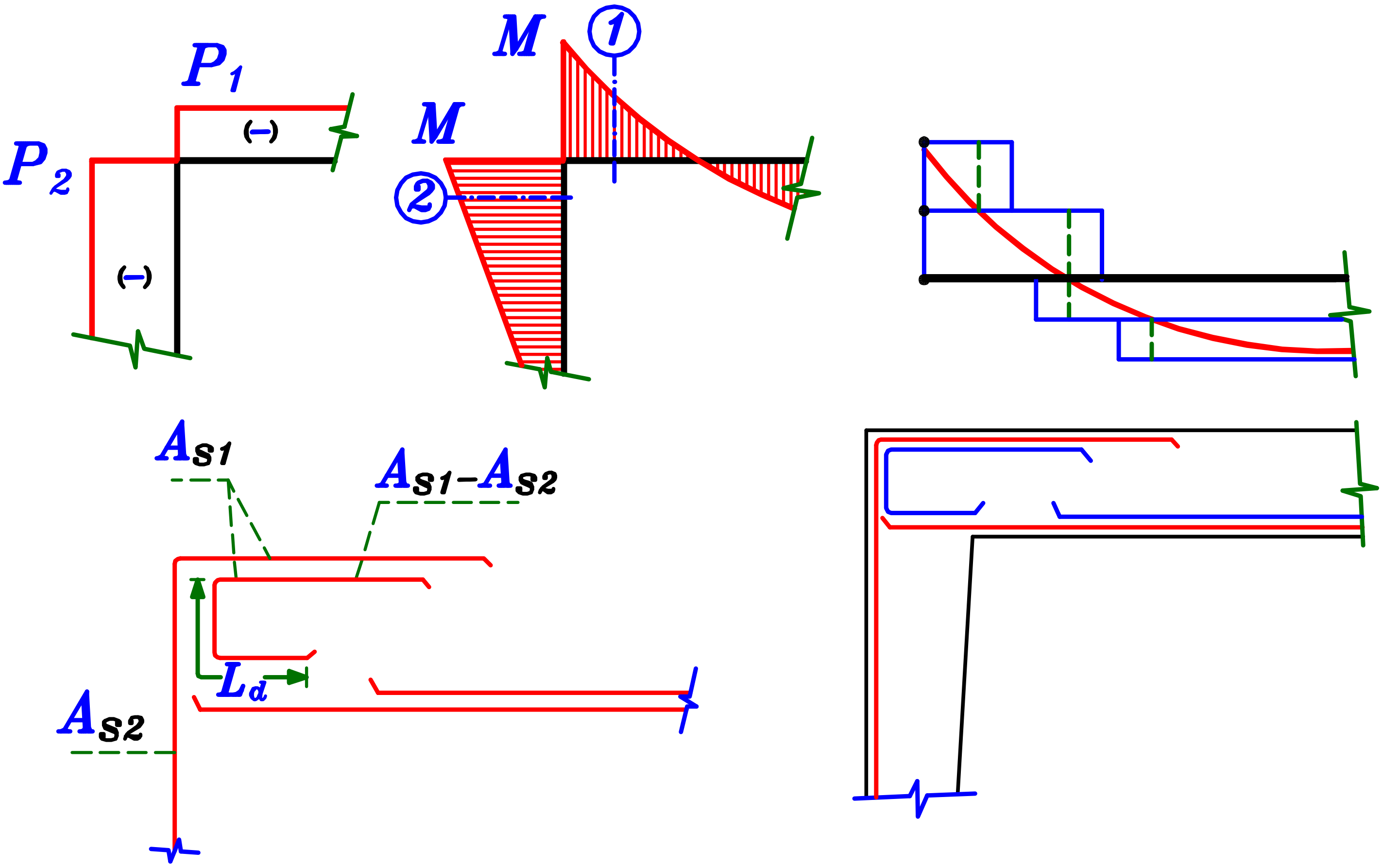


Fatal

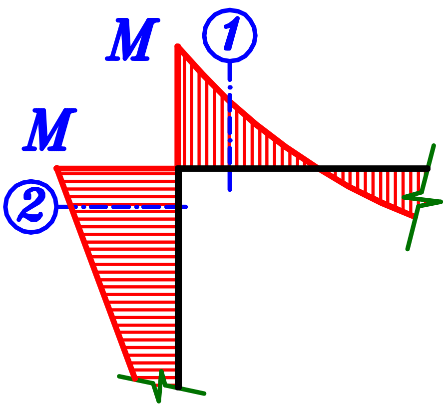
توقيف الحديد عند منطقة ال ***max. moment***

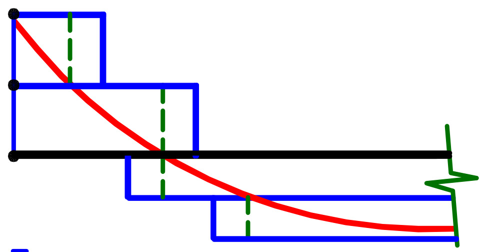
في حاله إختلاف كميه الحديد في قطاعين في نفس ال Joint

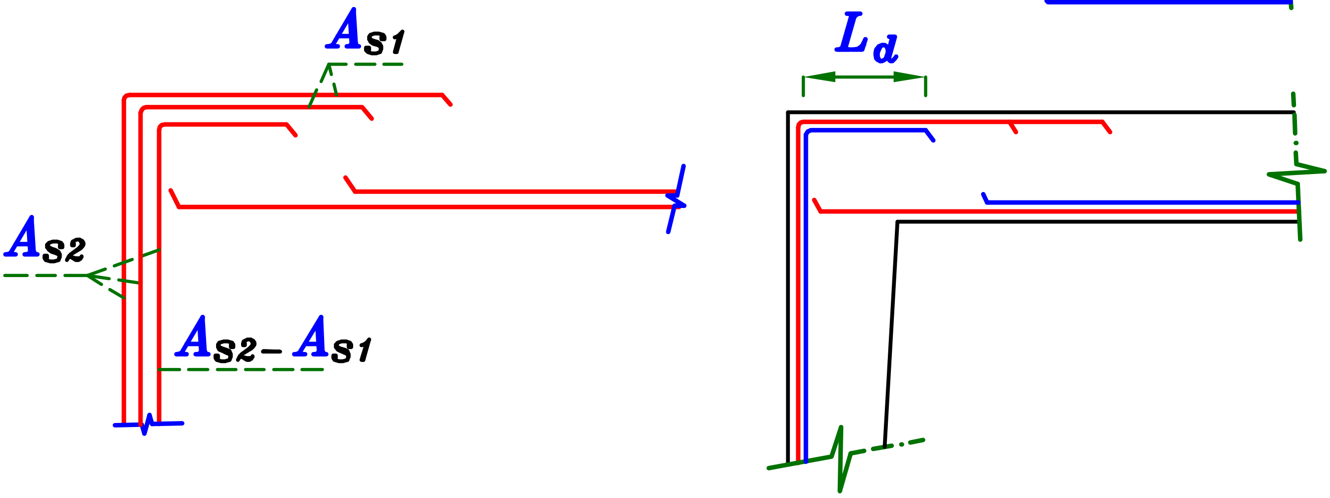
① تسليح الكمره أكبر من تسليح العمود .



② تسليح العمود أكبر من تسليح الكمره .





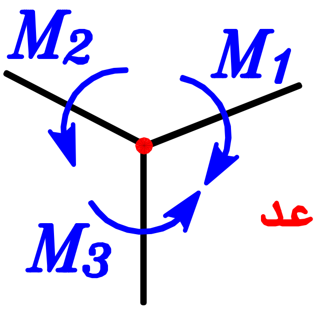


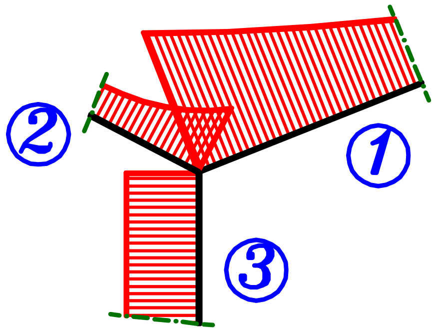
Joints with 3 members.

اذا كانت أسهم العزوم تدور فى نفس الاتجاه ⟵ لا توجد علاقه بين العزوم

اذا كانت أسهم العزوم تدور فى عكس الاتجاه ⟵ توجد علاقه بين العزوم تحدد من شكل ال *moment*

تباعد و تداخل





ملحوظه هامه

عندما تكون هناك علاقه بين عزمين يجب أن ننظر الى **B.M.D.** لكى نحدد اذا كانت هذه العلاقه تباعد أم تداخل .

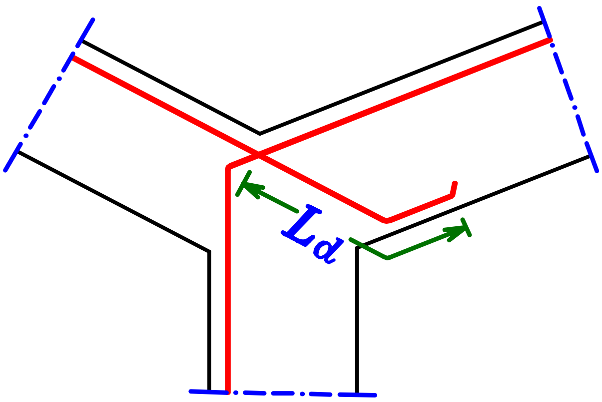
$$M_1 = M_2 + M_3$$

M_1 , M_2 ⟶ تداخل

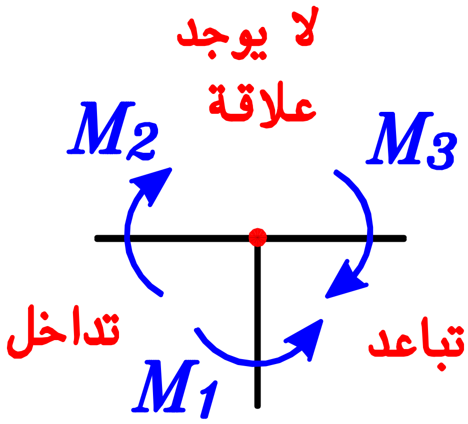
M_1 , M_3 ⟶ تباعد

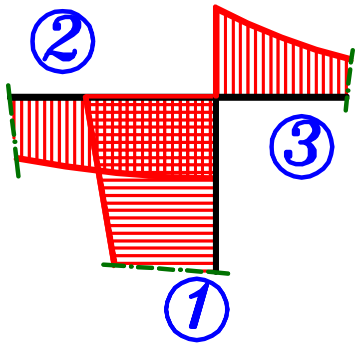
M_2 , M_3 ⟶ لا يوجد علاقه

إذا وجد تباعد و تداخل معاً نكمل حديد التباعد أولاً ثم نعمل مقص التداخل .



تباعد و تداخل



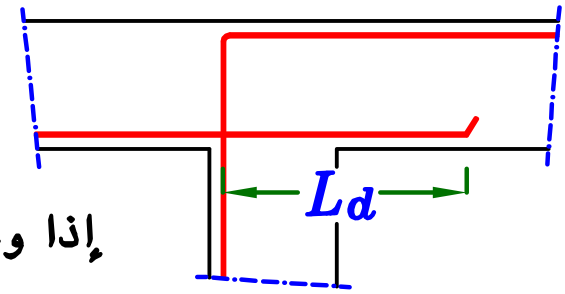


$$M_1 = M_2 + M_3$$

$M_1 , M_2 \longrightarrow$ تداخل

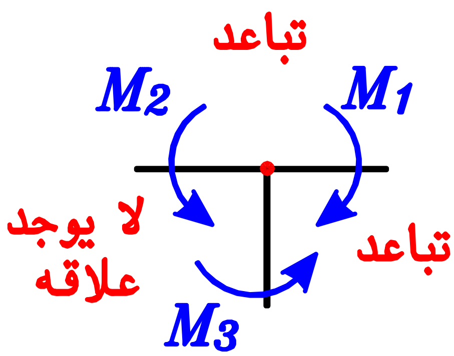
$M_1 , M_3 \longrightarrow$ تباعد

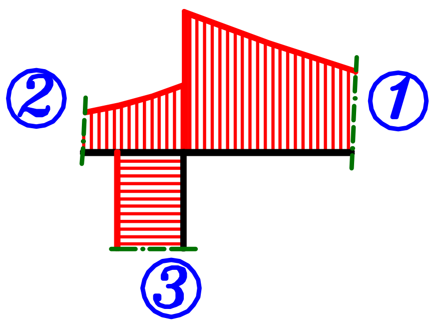
$M_2 , M_3 \longrightarrow$ لا يوجد علاقه



إذا وجد تباعد و تداخل معاً نكمل حديد التباعد أولاً ثم نعمل مقص التداخل .

تباعد و تباعد



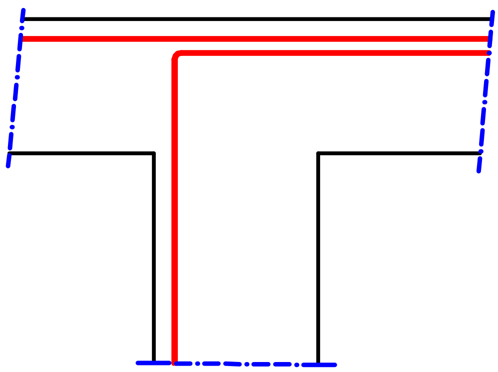


$$M_1 = M_2 + M_3$$

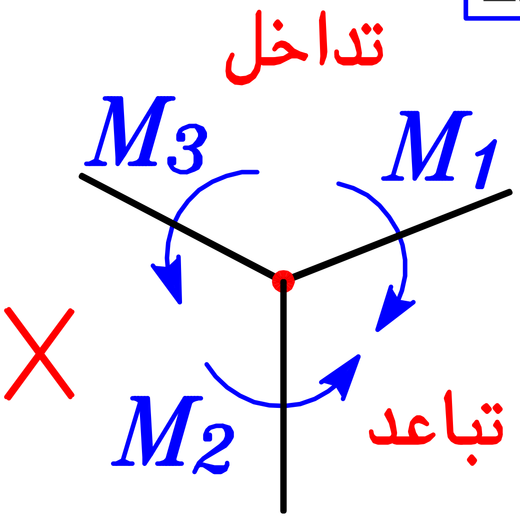
$M_1 , M_2 \longrightarrow$ تباعد

$M_1 , M_3 \longrightarrow$ تباعد

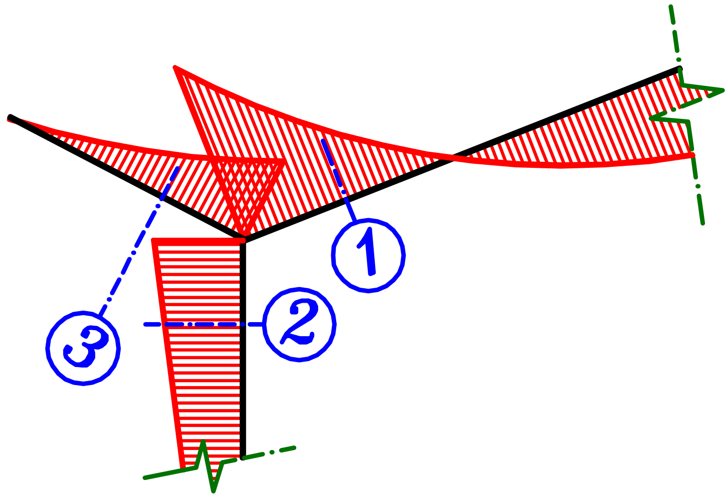
$M_2 , M_3 \longrightarrow$ لا يوجد علاقه



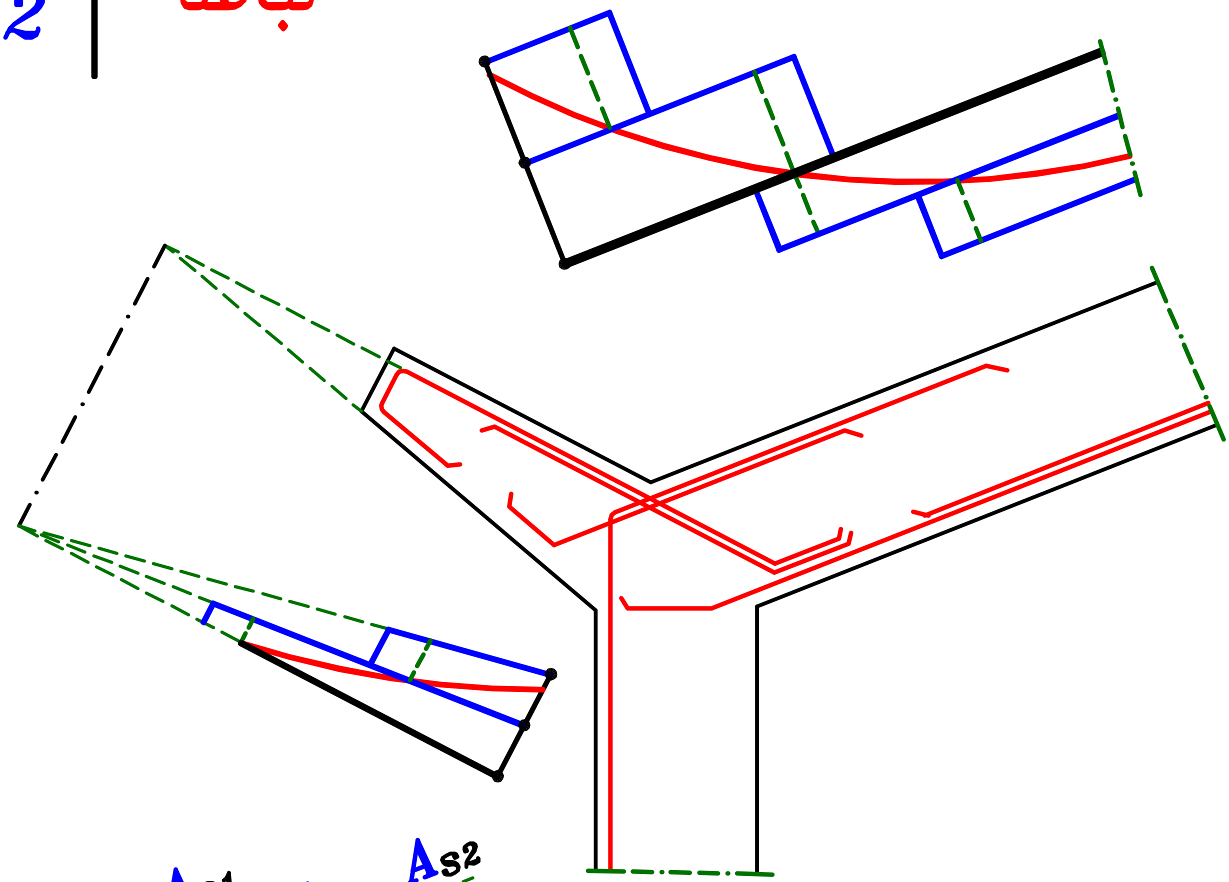
إذا وجد تباعد و تباعد نكمل الحديد الموجود على إستقامه واحده أولاً .
ثم نعمل التباعد الاخر صف ثانى .

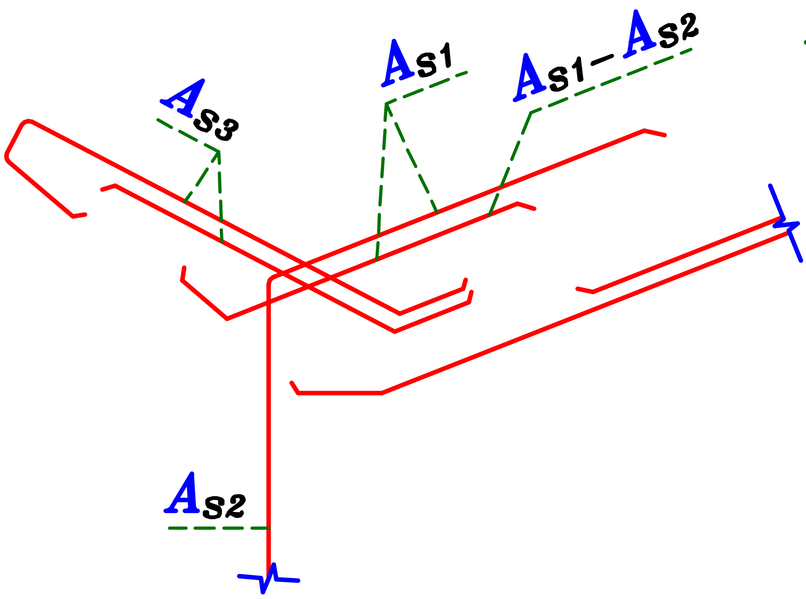


تداخل
M3
M1
M2
تباعد

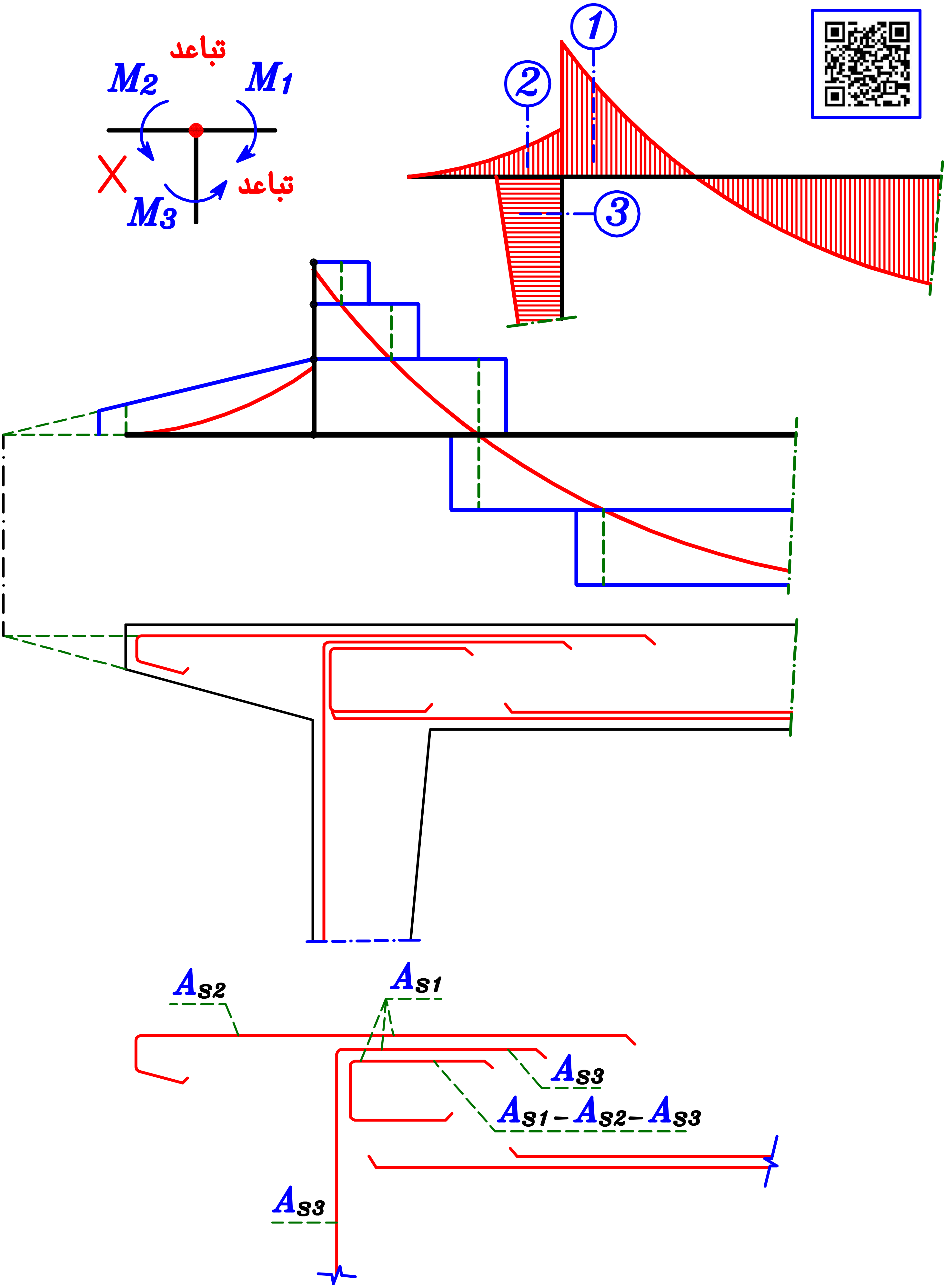


1
2
3



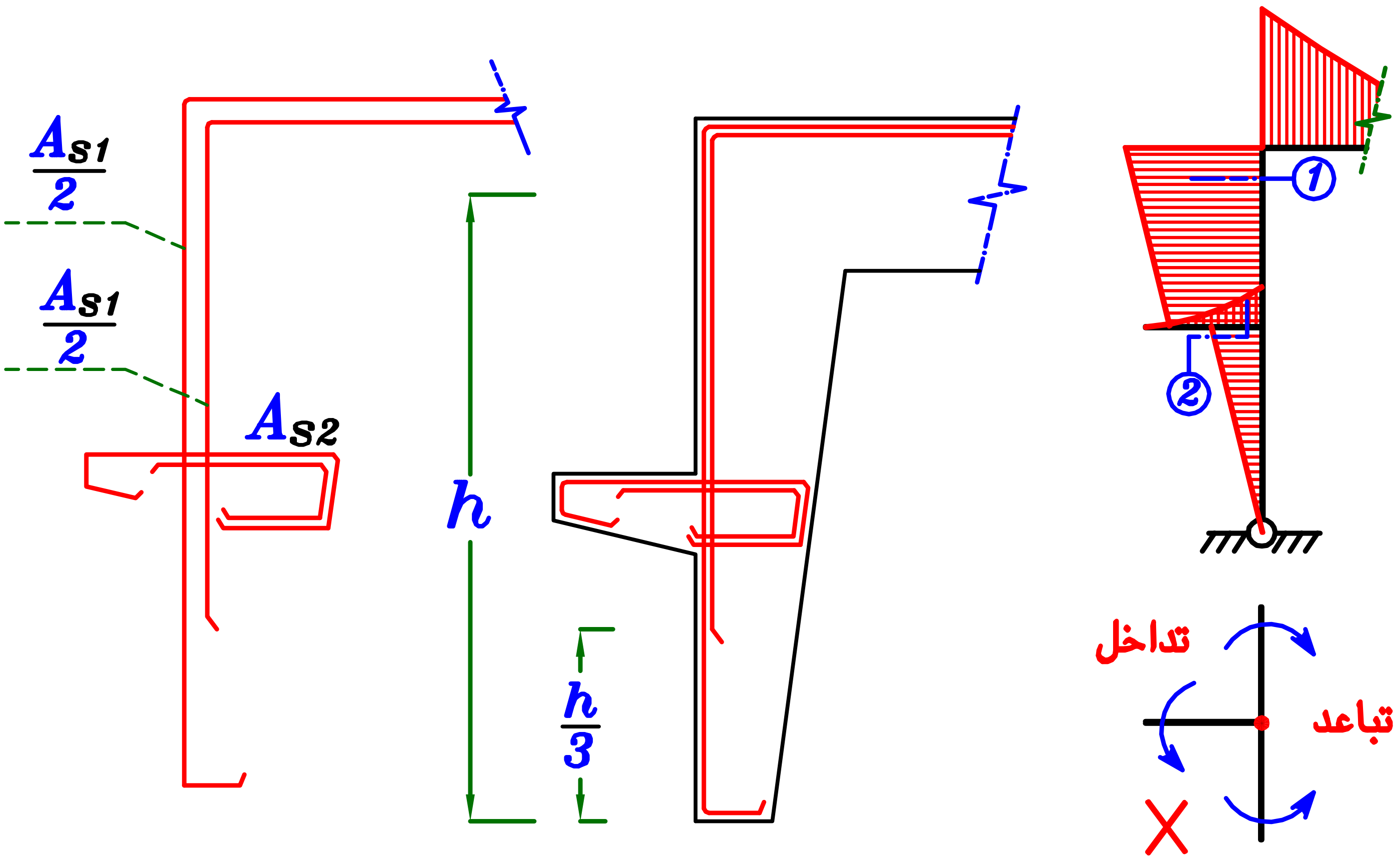


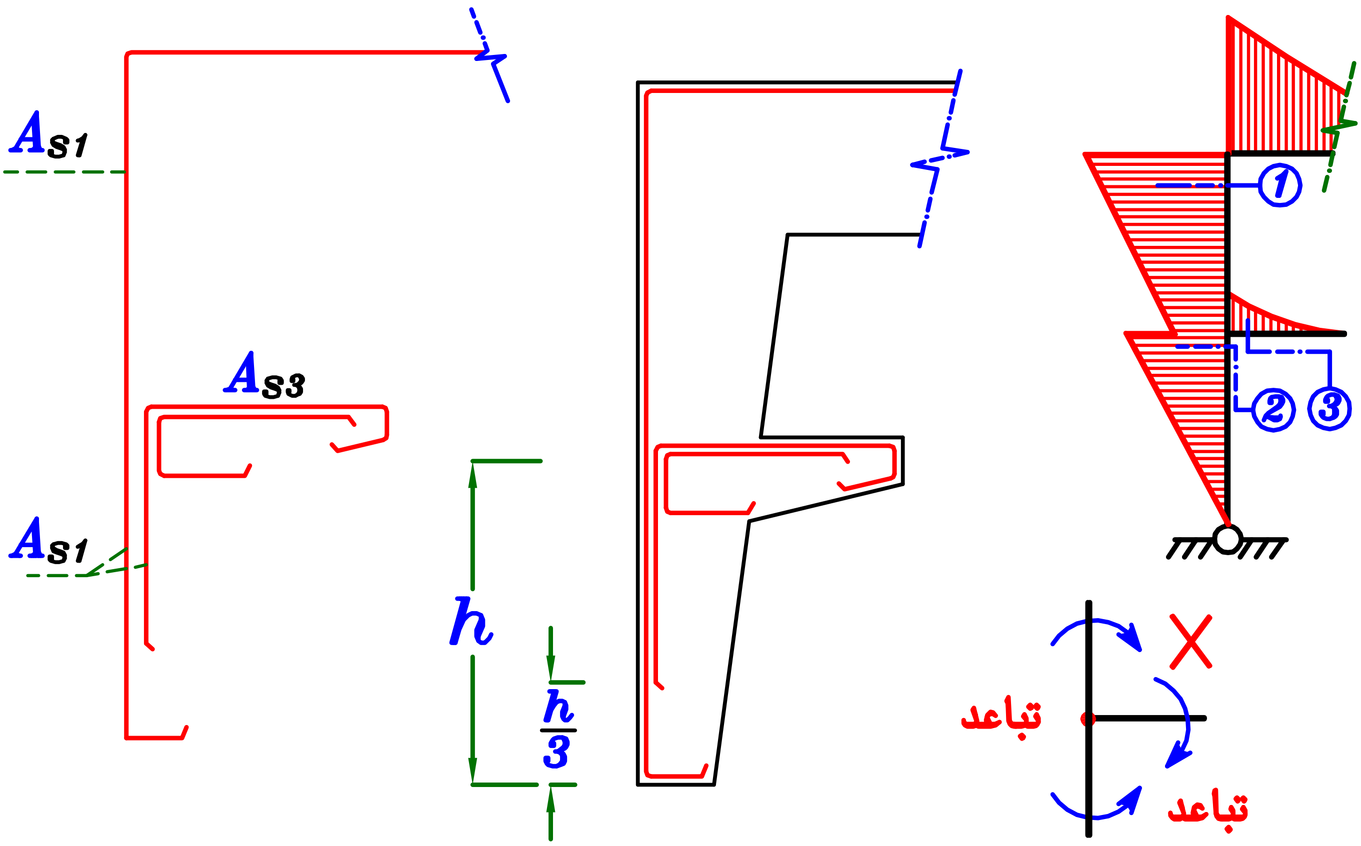
As3
As1
As1-As2
As2

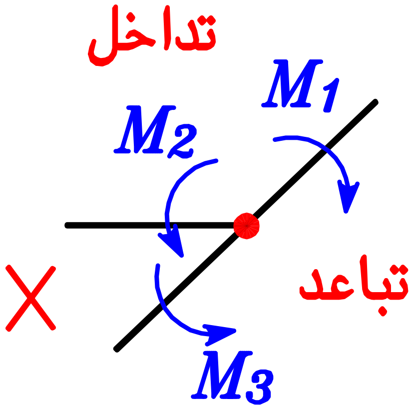


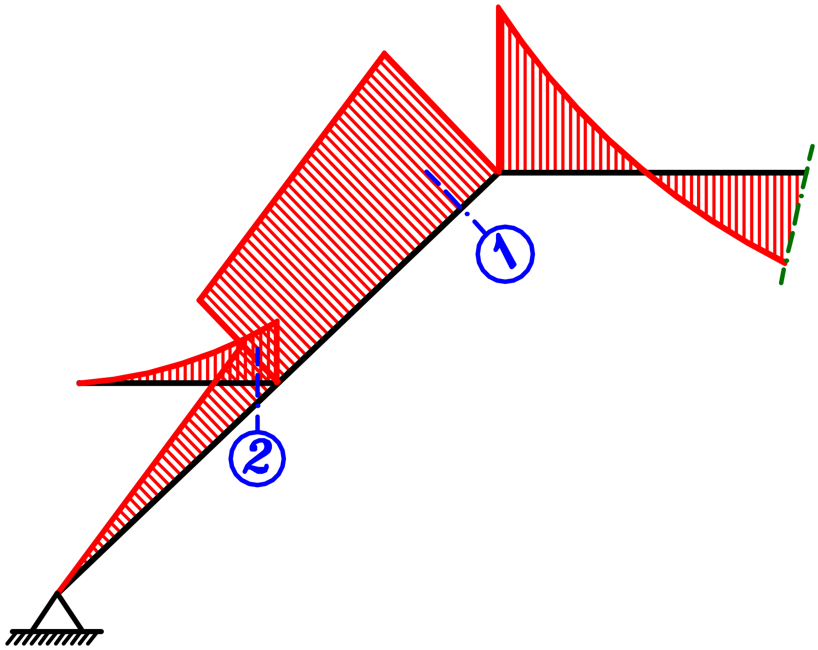
تباعد
M2
M1
تباعد
M3
1
2
3
As2
As1
As3
As1 – As2 – As3
As3

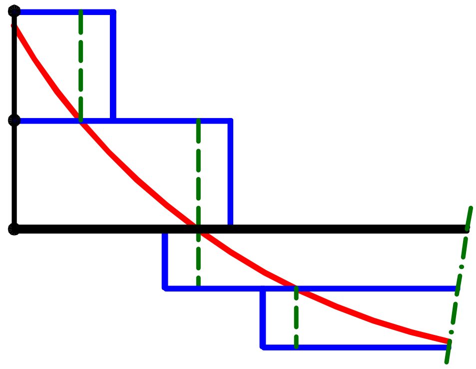
Frequent Joints.

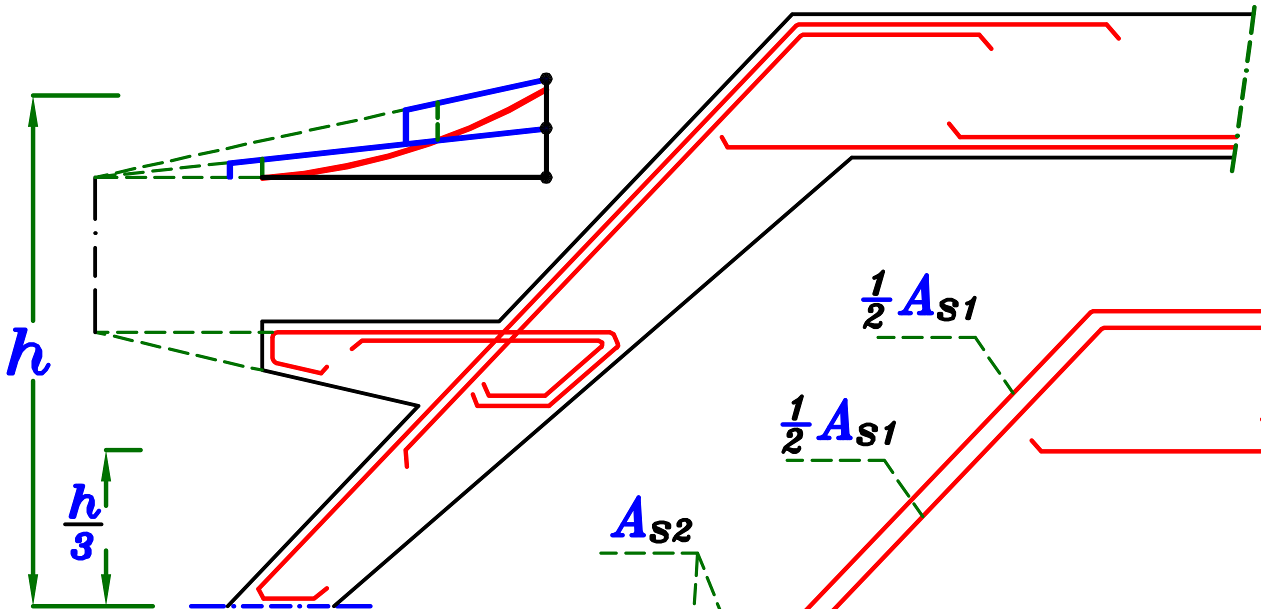


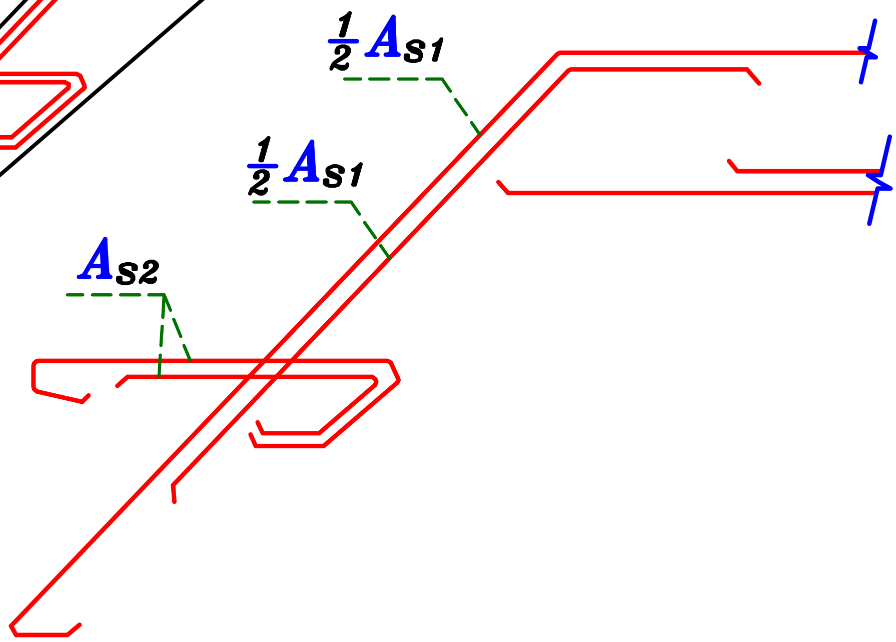


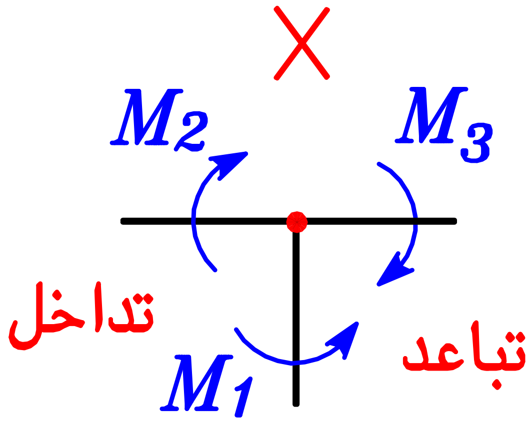

تداخل
M_1
M_2
تباعد
M_3


1
2

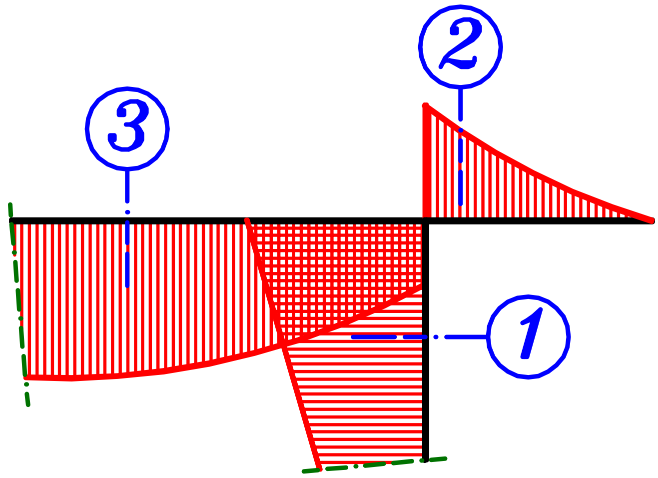



h
$\frac{h}{3}$

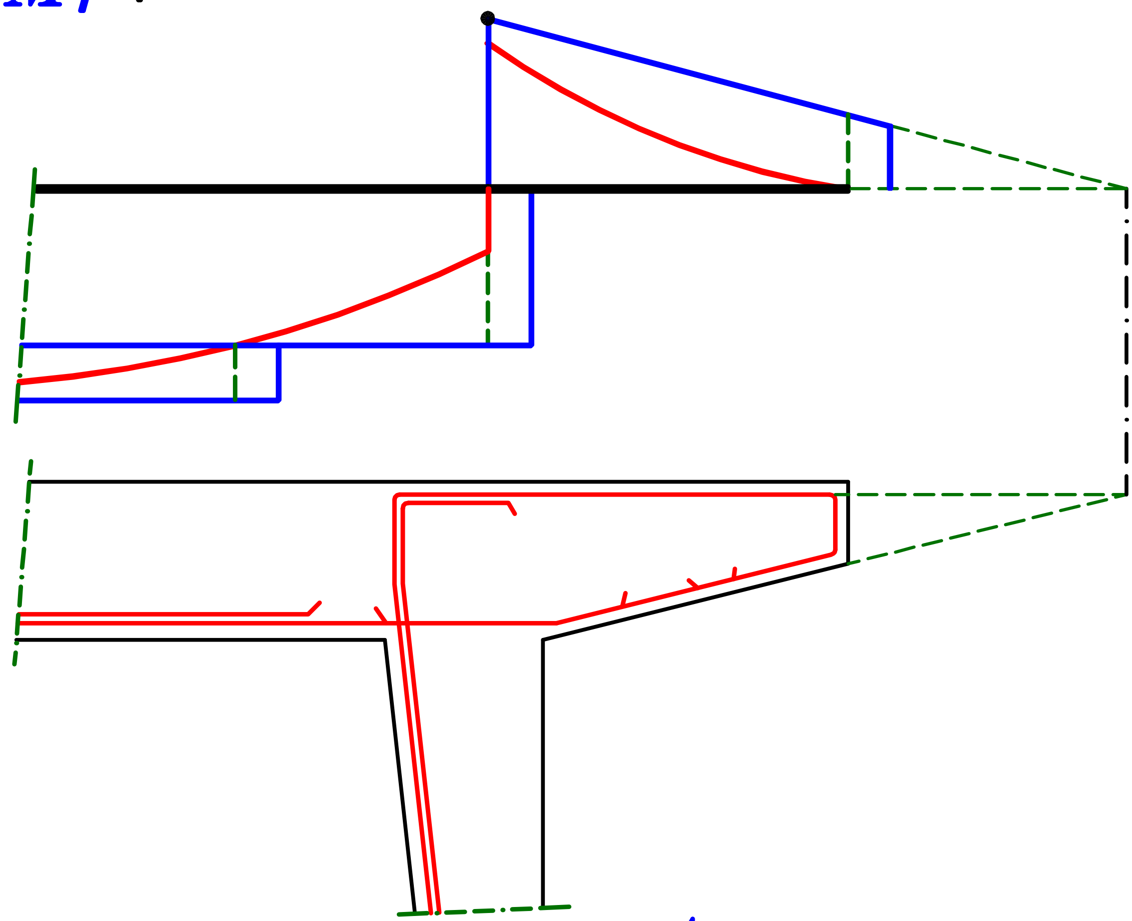

$\frac{1}{2} A_{s1}$
$\frac{1}{2} A_{s1}$
A_{s2}

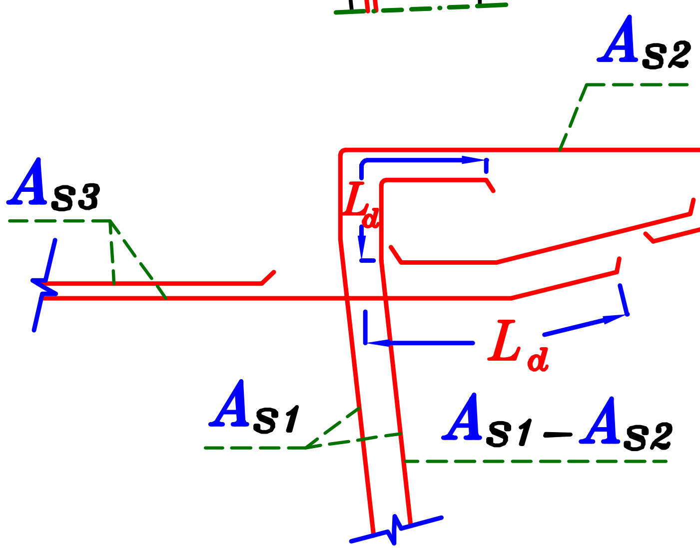


M_2
M_3
تداخل
تباعد
M_1



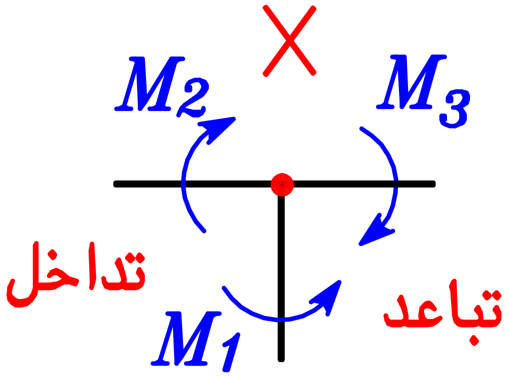
3
2
1

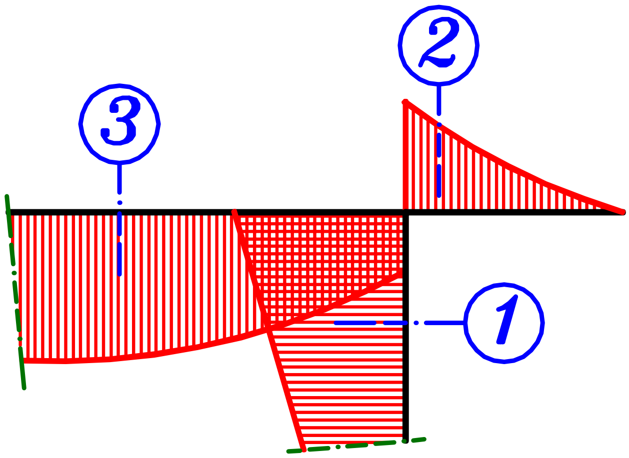


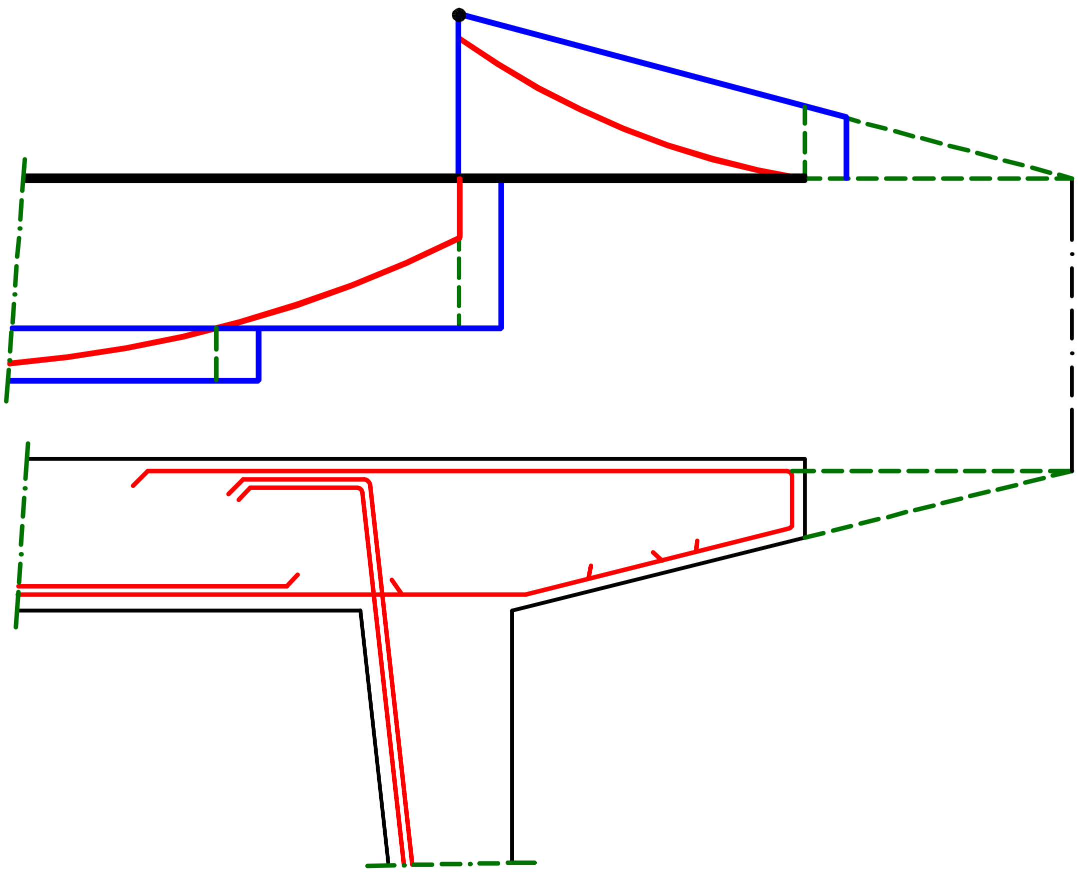


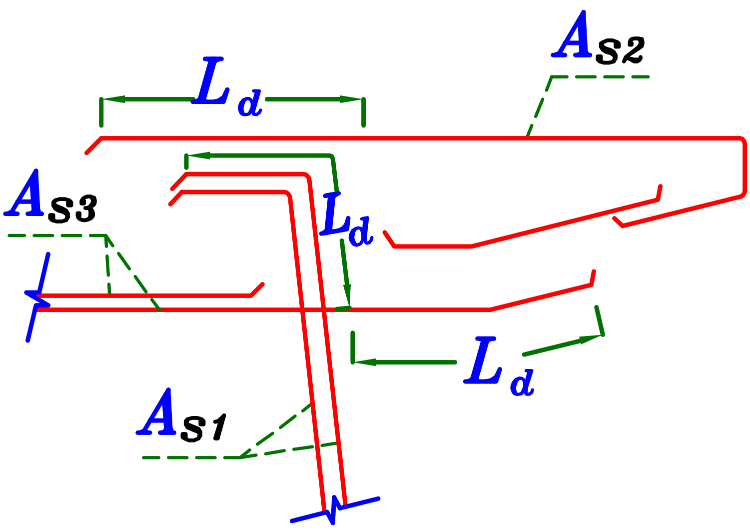
A_{s2}
A_{s3}
L_d
L_d
A_{s1}
$A_{s1} - A_{s2}$

Another Concept of RFT.

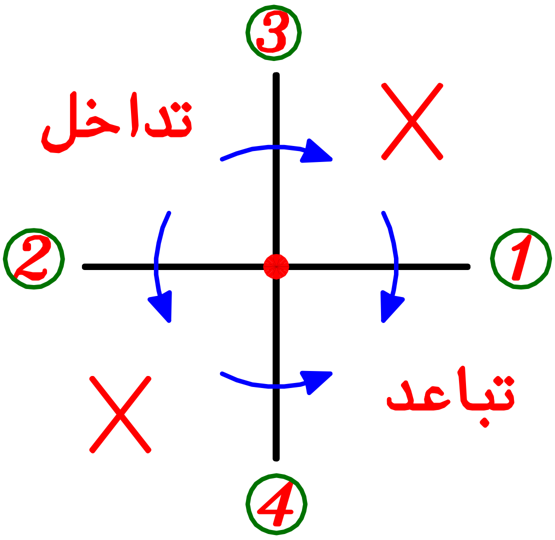


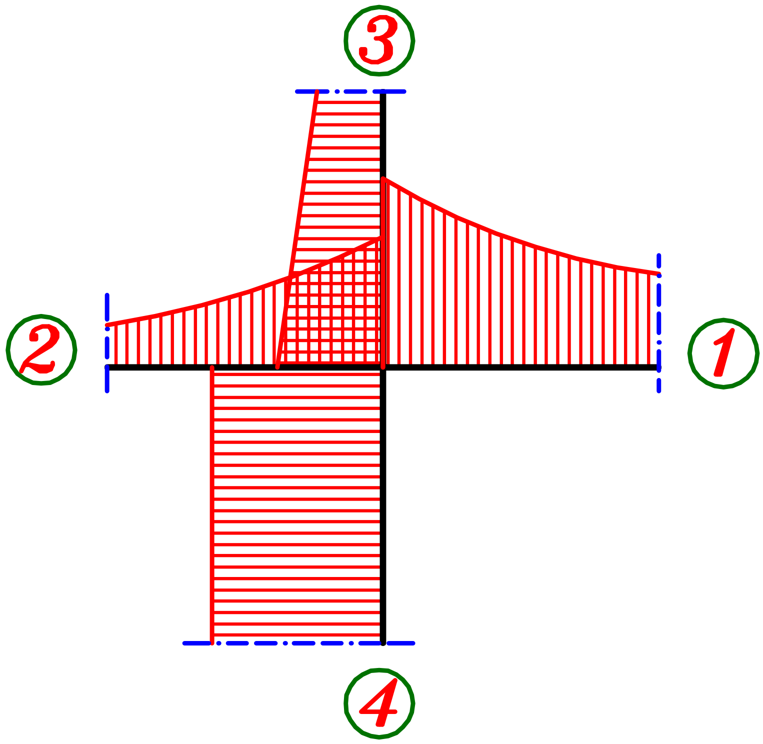


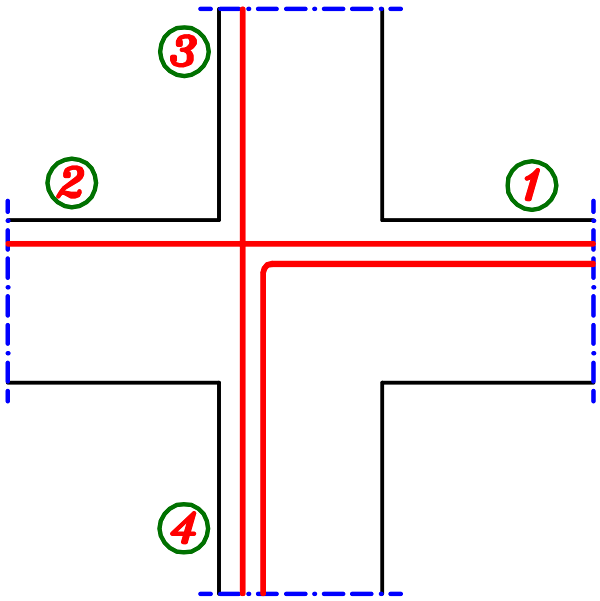




RFT. of Four mrmber Joint.







بين 1 و 2 يوجد تباعد

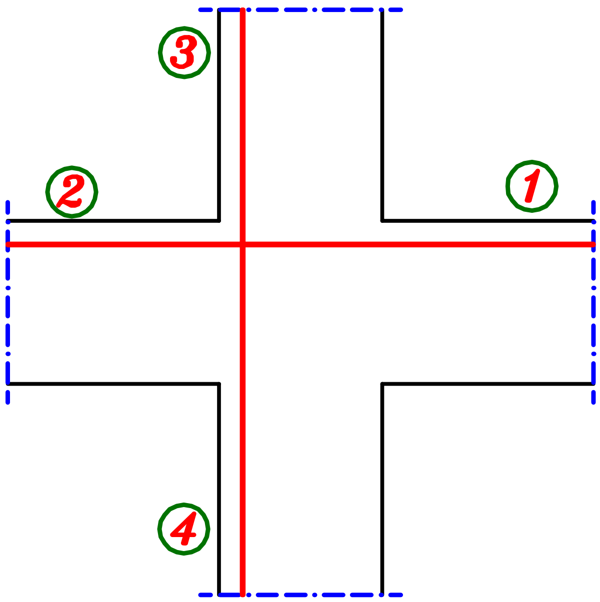
بين 3 و 4 يوجد تباعد

بين 1 و 4 يوجد تباعد

بين 2 و 3 يوجد تداخل

بين 1 و 3 لا يوجد شيئ

بين 2 و 4 لا يوجد شيئ



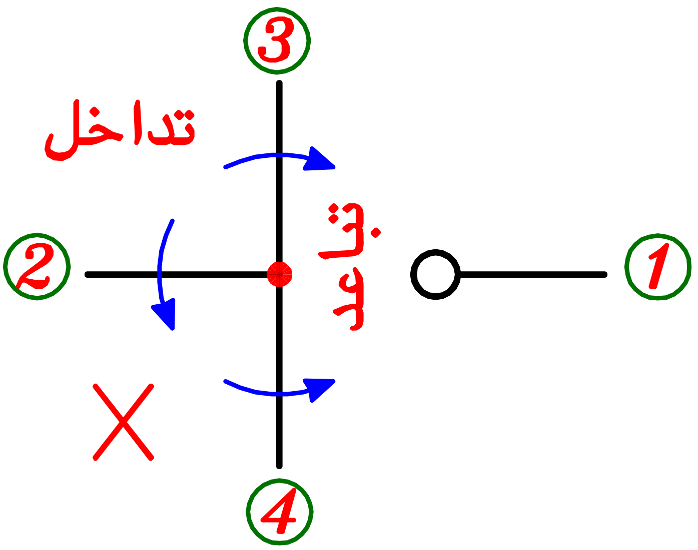
بين 1 و 4 يوجد تباعد

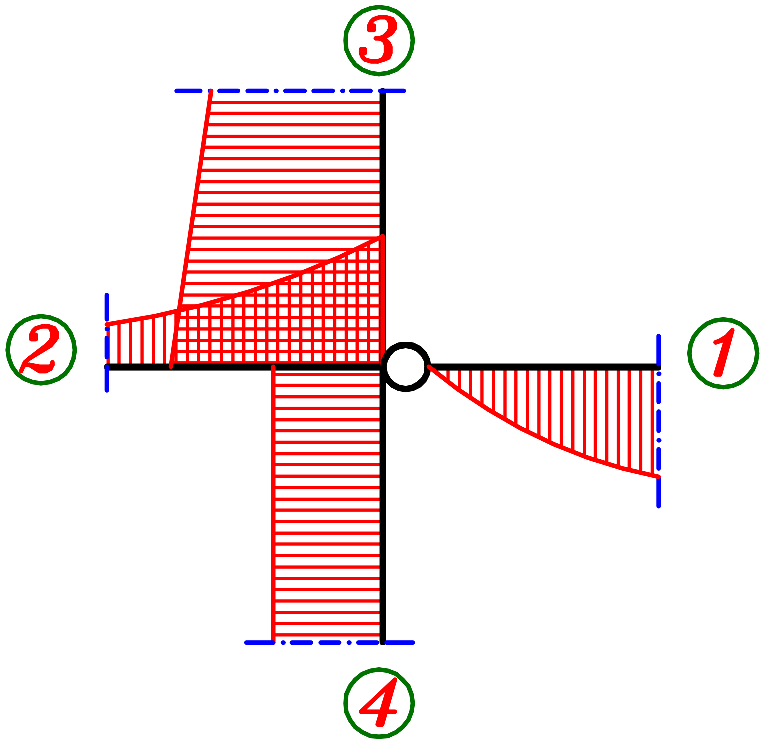
لكن اذا لم يحتاج اى *member* منهم

حديد من ال *member* الاخر

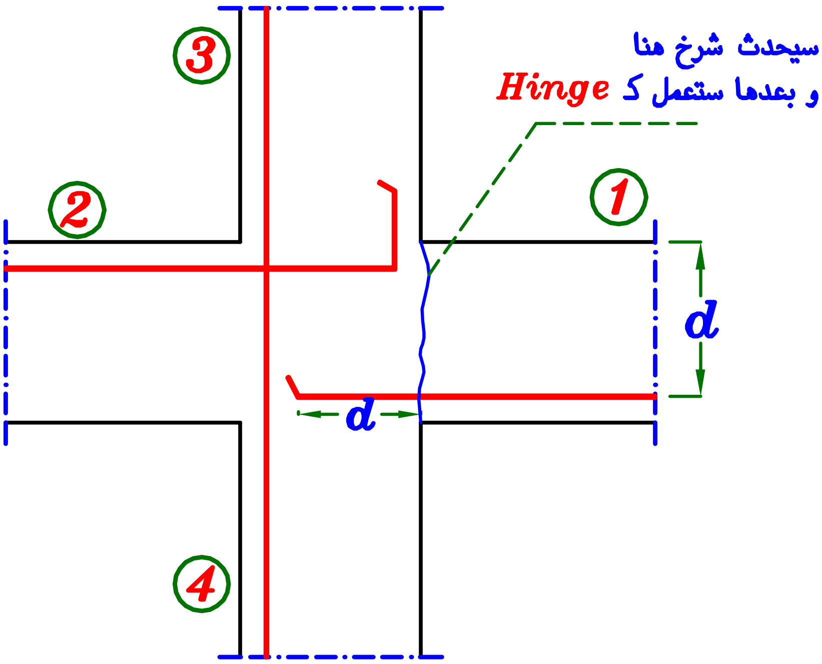
ممكن ان لا نكمل بينهم اى اسياخ مشتركه

RFT. of Four mrmber Joint with Hinge.





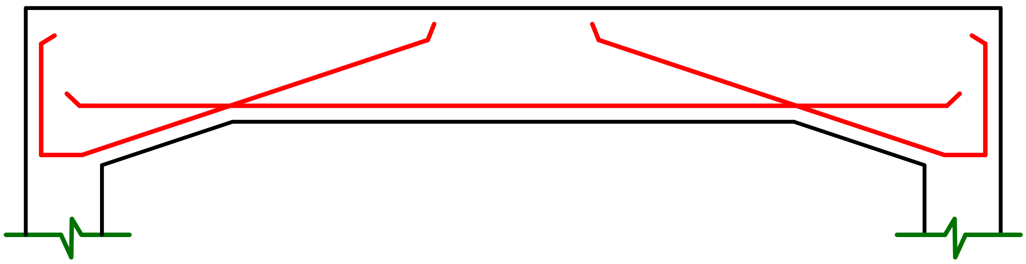
فى هذه الحاله يتم تنفيذ الـ **Hinge** بأن لا نكمل الحديد من **member** 1
الى داخل الـ **Joint** الا قيمه صغيره بمقدار d الكمره



بين 3 و 4 يوجد تباعد
بين 2 و 3 يوجد تداخل
بين 2 و 4 لا يوجد شيئ

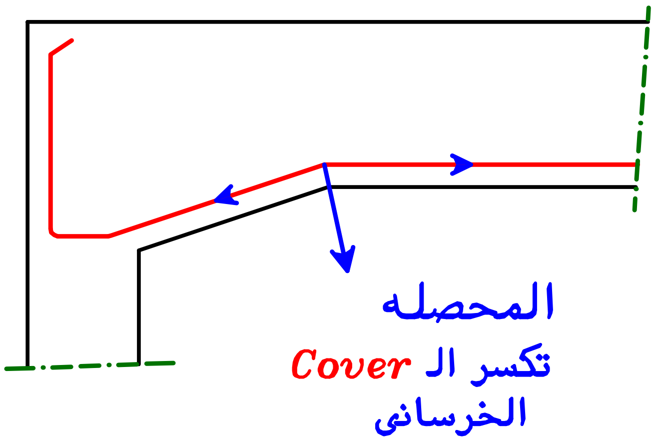
و تعتبر الـ **Joint** عند التسليح كأنها **3 member joint** تباعد و تداخل

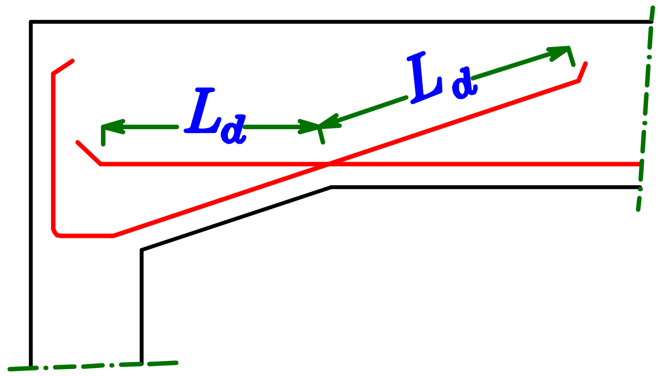
Variable Depth.

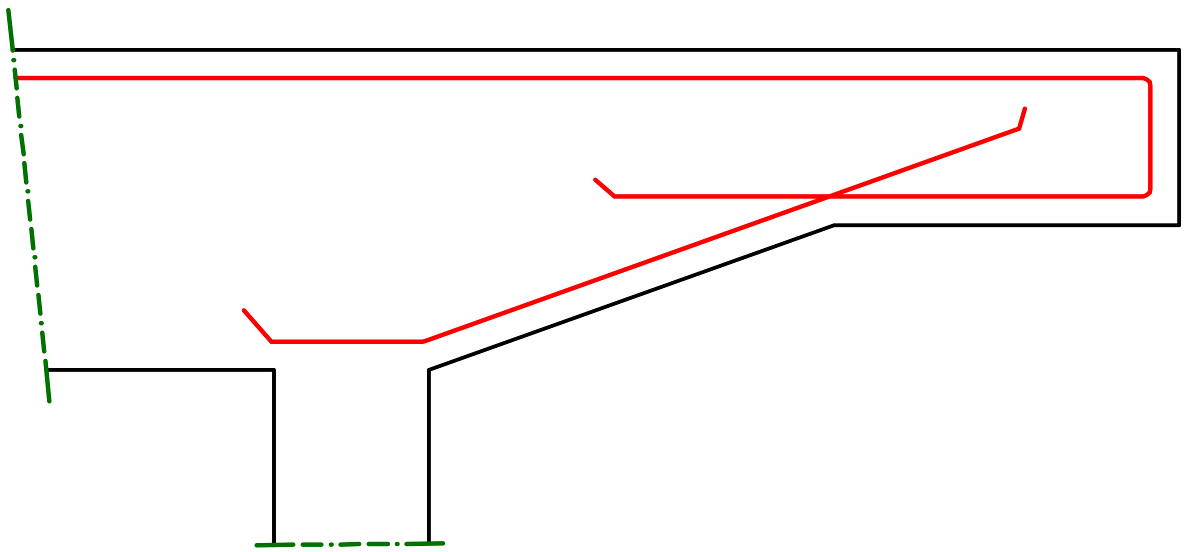


Fatal.

مقص

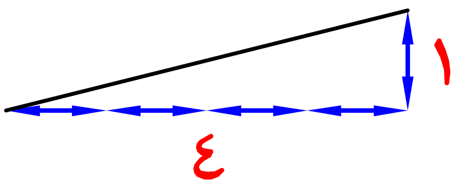




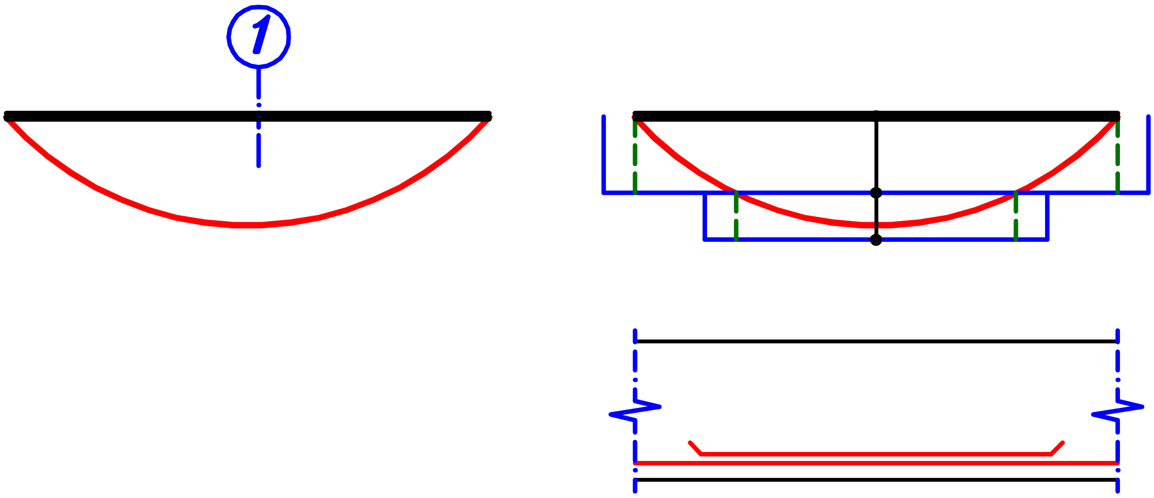


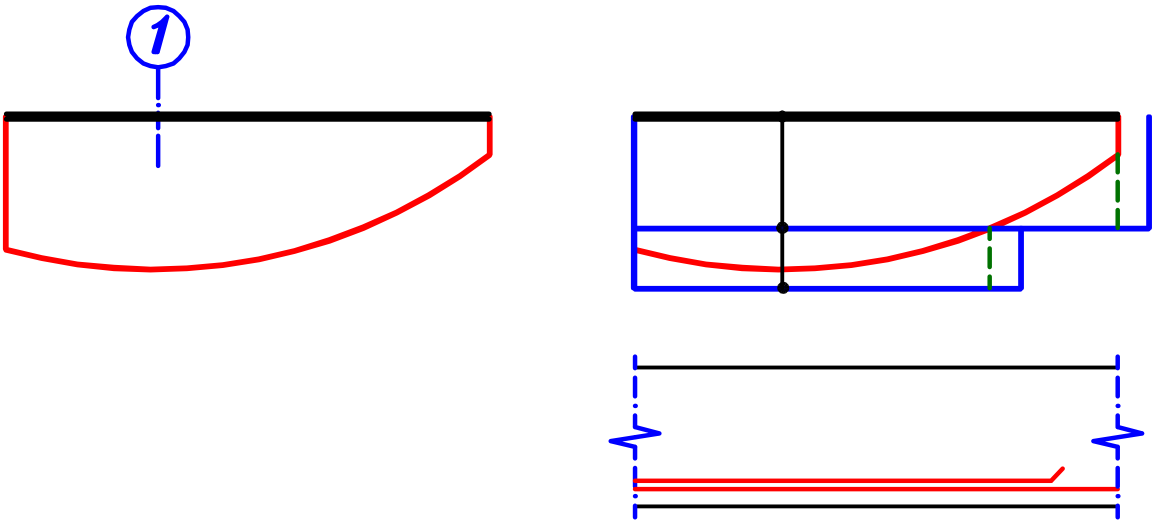
اذا كان ميل الخرسانه غير معطى فى المسأله

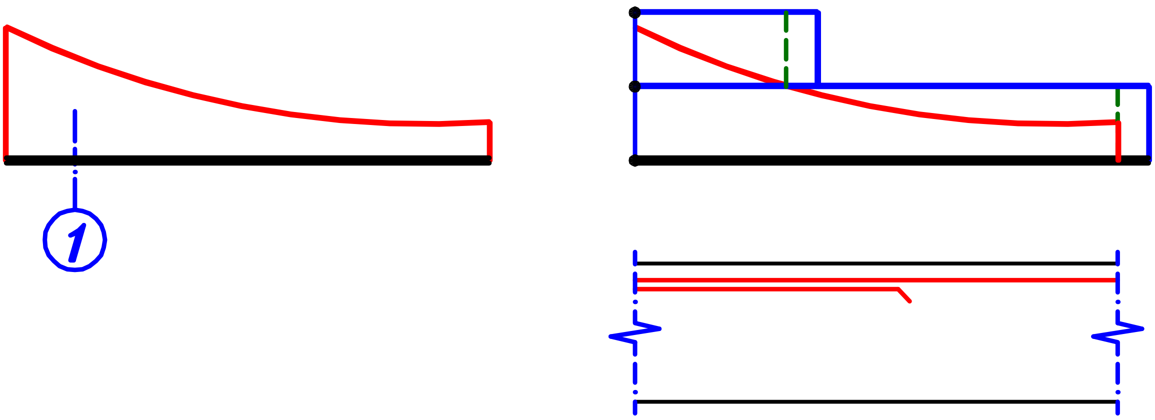
يفضل أن نأخذه بميل ١ : ٤



Critical Sections & Details of RFT.
For different members

1


2


3


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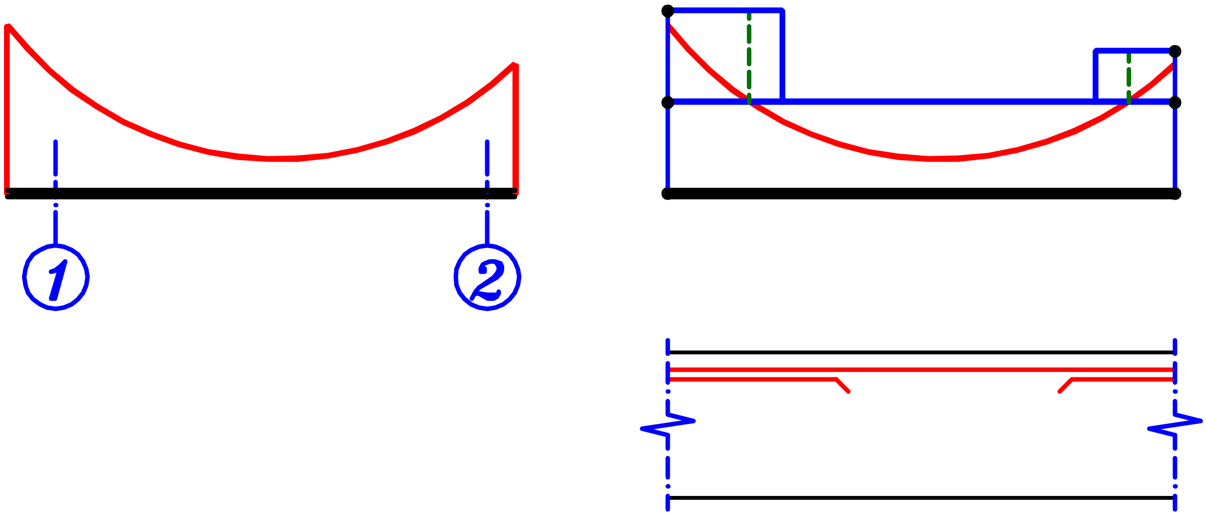

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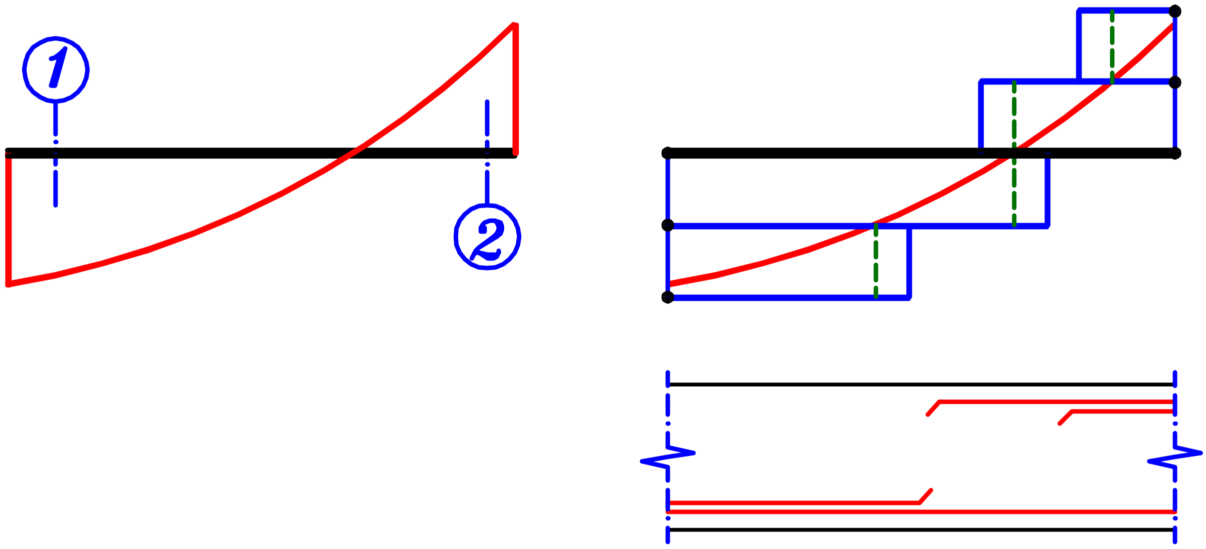

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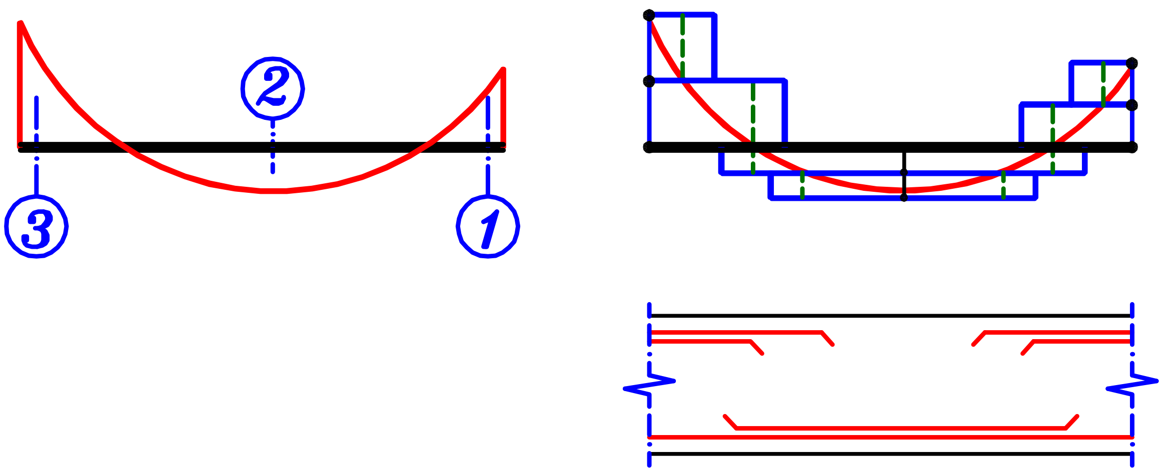
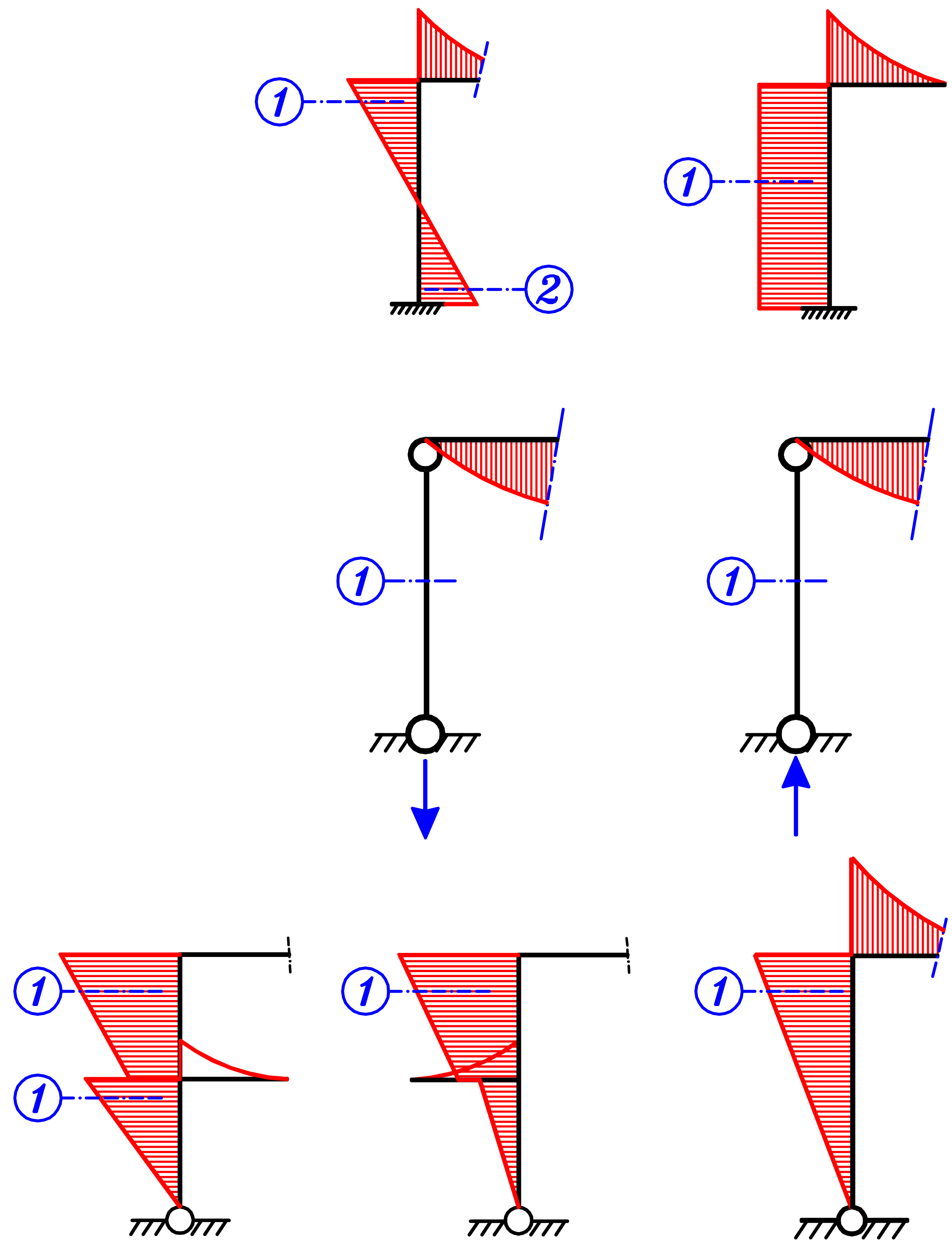


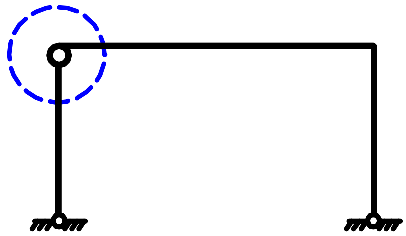

1
2
1
1
1
1
1
1
1

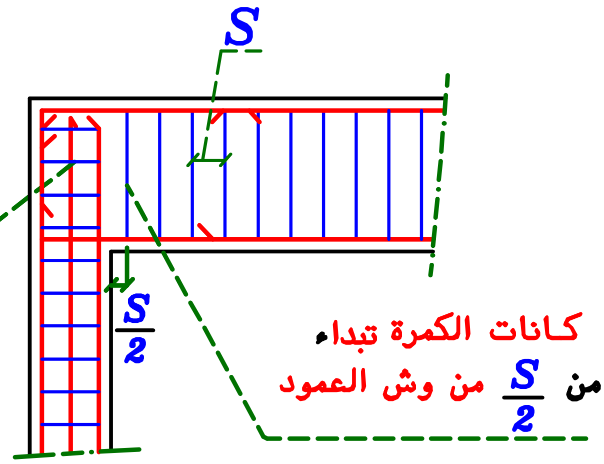
Stirrups الكانات

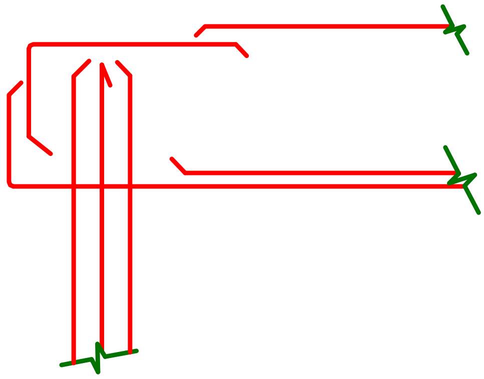
ترسم الكانات دائما عموديه على ال .C.L

① **Hinged Joints.** (Joint between the beam & Link member)

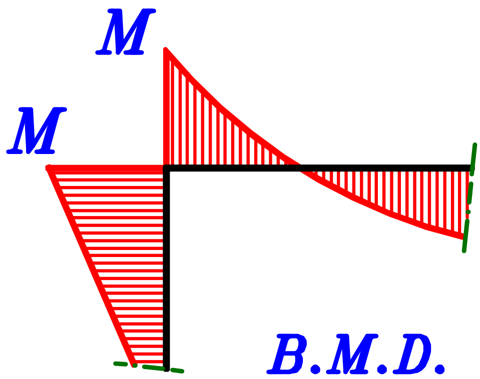
joint لا تنتقل عزوم



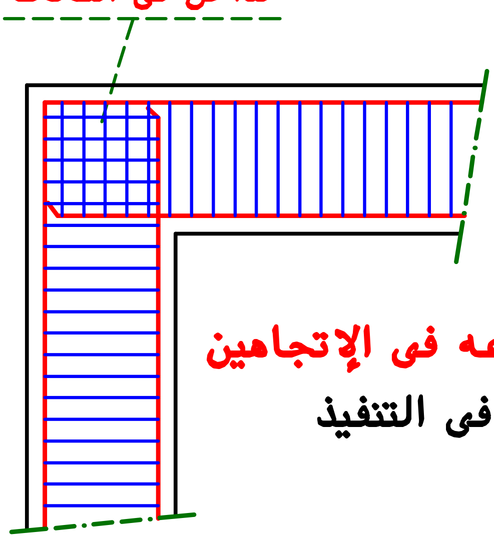




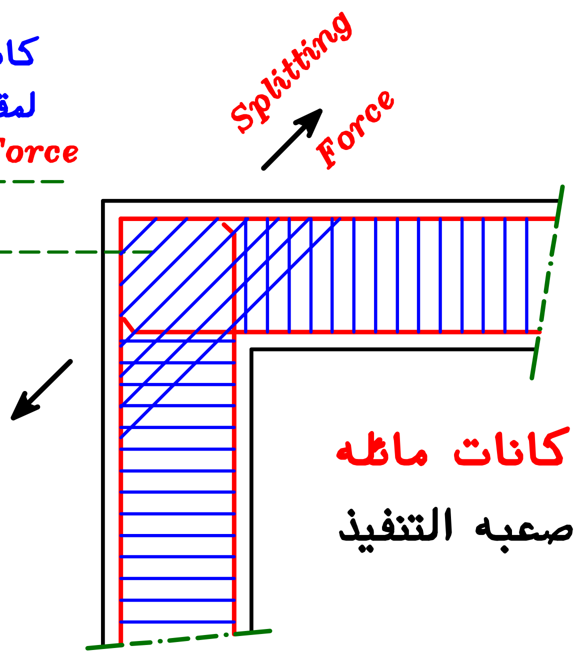
② **Rigid Joints.** joints تنقل عزوم
تقاطع أو تداخل



> يجب أن تكمل الكانات فى الأتجاهين
> أى يحدث تداخل فى الكانات .

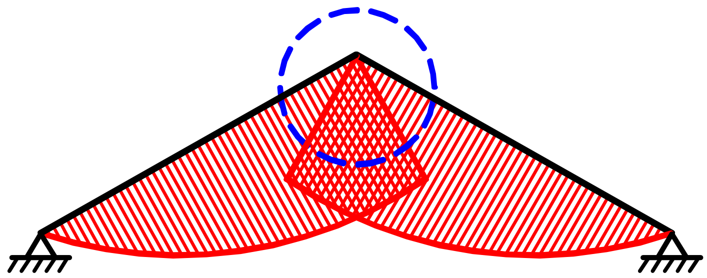


كانات متقاطعه فى الإتجاهين
مفضله فى التنفيذ

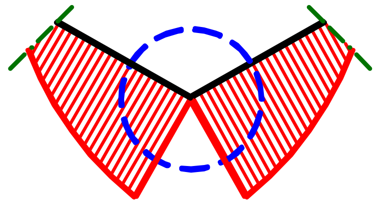


كانات مائله
صعبه التنفيذ

Joints تنتقل عزوم سواء تداخل أو تباعد .



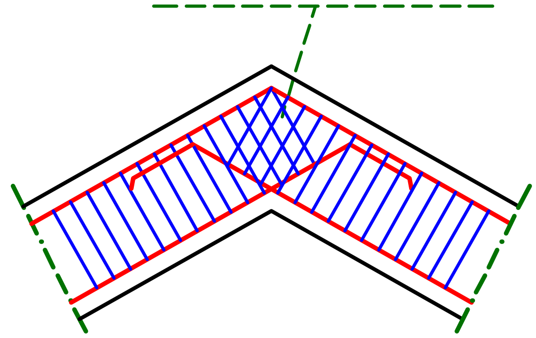
تداخل فى الـ *B.M.*



تباعد فى الـ *B.M.*

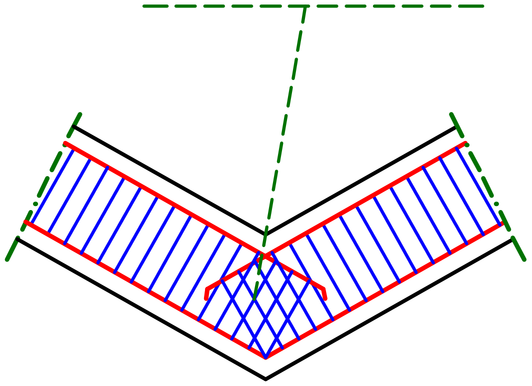
ترسم الكانات دائما عموديه على الـ *C.L.*

تداخل فى الكانات



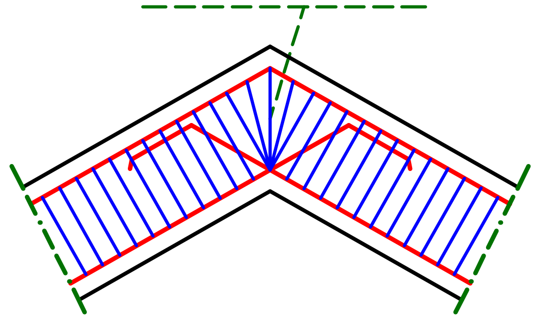
كانات متقاطعه فى الإتجاهين ✓✓
مفضله فى التنفيذ

تداخل فى الكانات



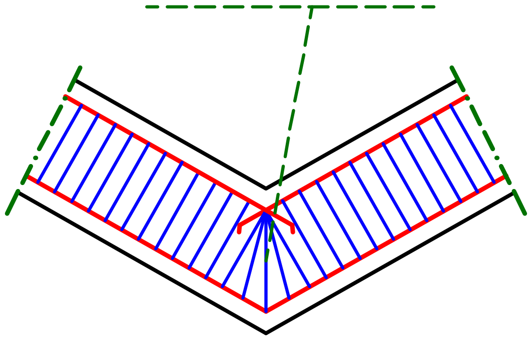
كانات متقاطعه فى الإتجاهين ✓✓
مفضله فى التنفيذ

كانات مروحه



كانات مروحه
صعبه التنفيذ

كانات مروحه

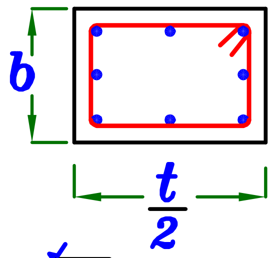


كانات مروحه
صعبه التنفيذ

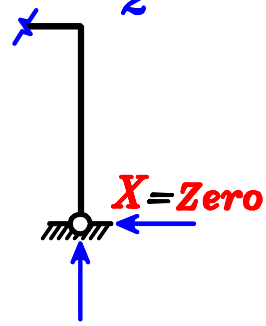
Cross Sections.

ⓐ Axially Loaded Member.

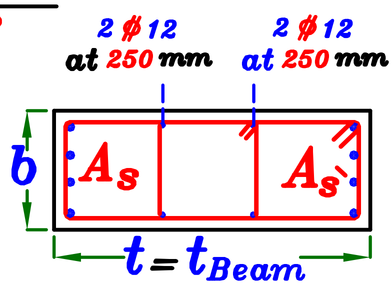
① Link Member. (Compression OR Tension) $(b * \frac{t}{2})$



② فى بعض الاحيان يكون العمود معطى كما بالشكل
و لكن لا يكون عليه B.M.
فى هذه الحالة تؤخذ تخانته بنفس تخانه الكمره المحموله عليه

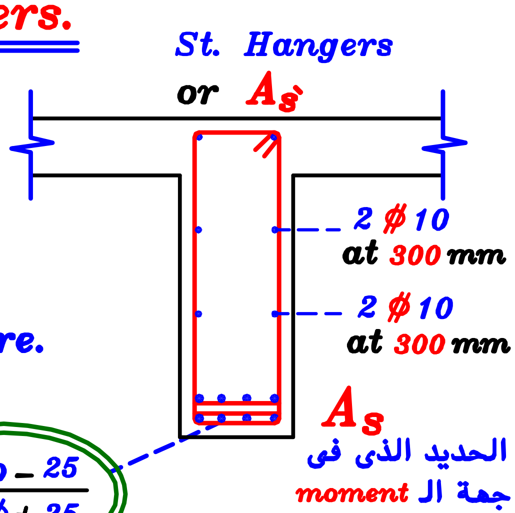


$$A_{s\,min.} = \frac{0.8}{100} * b * t \qquad A_s = A_{s`} = \frac{A_{s\,min.}}{2}$$



ⓑ Horizontal & Inclined members.

لا توجد كانات داخليه (مثل الكمرات)



الحديد الذى فى الجهه المقابله لل moment

Stirrup Hangers IF Ten. Failure.

Compression Steel $(A_{s`})$ IF Comp. Failure.

$$n = \frac{b - 25}{\phi + 25}$$

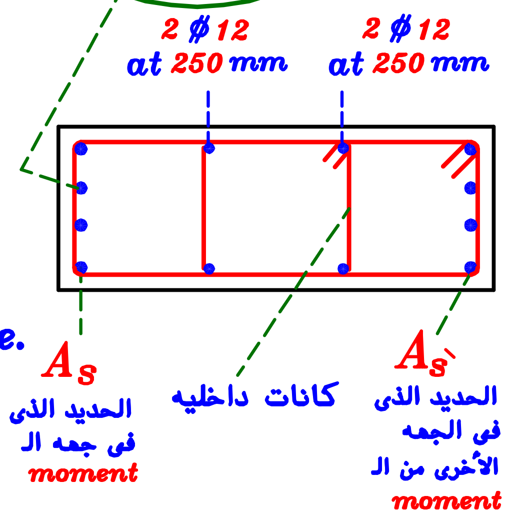
و غالبا فى الكمرات تكون $\frac{e}{t} > 0.5$

ⓒ Vertical members. مثل الأعمده

$$n = \frac{b - 25}{\phi + 25}$$

أكبر مسافه بين سيخين متتاليين = ٢٥٠ مم

أكبر مسافه بين فرع كانة و الأخر = ٣٠٠ مم

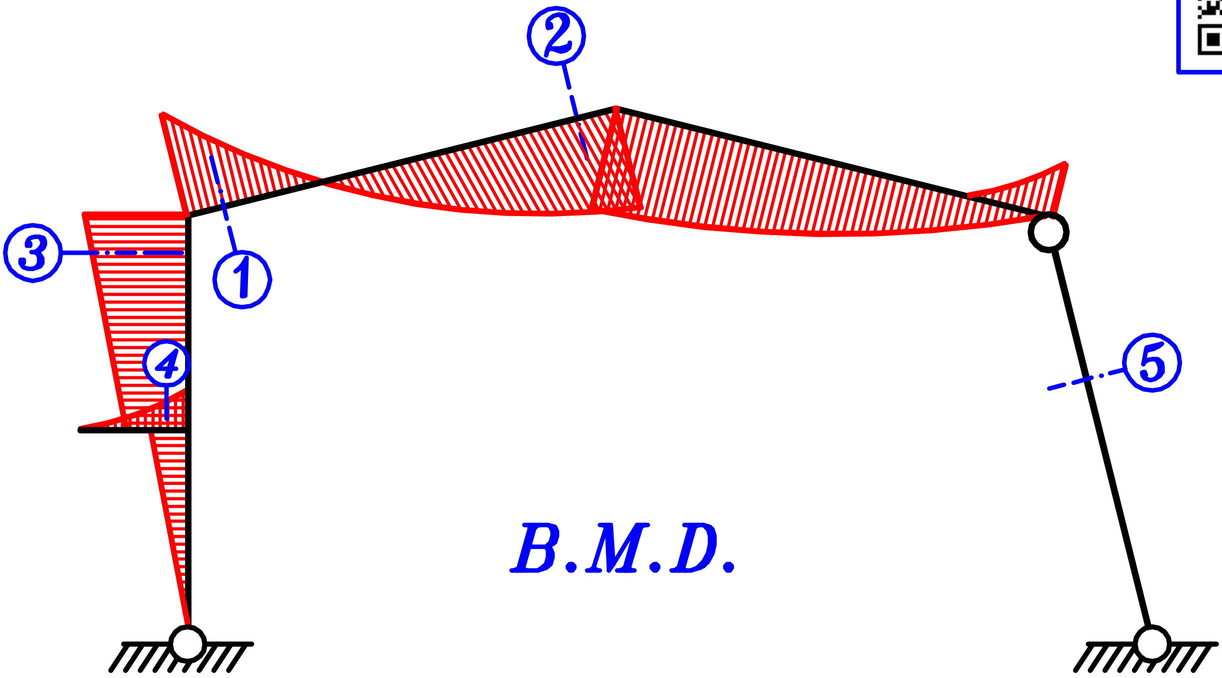


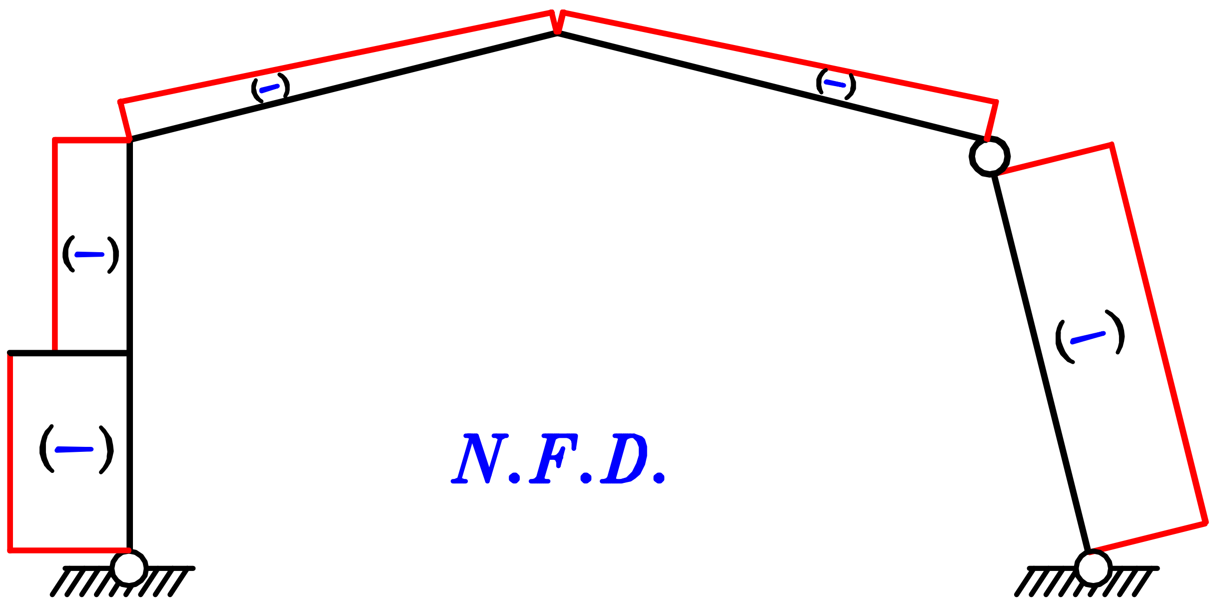
الحديد الذى فى الجهه الأخرى من ال moment

Take $A_{s`}$ = Stirrup Hangers $\simeq 0.4 A_s$ IF Ten. Failure.

Take $A_{s`} = A_s$ IF Comp. Failure.

Steps to design a Frame.





بعد رسم ال **B.M.D. & N.F.D.**

نحدد ال ***Critical Sections*** كما سبق .

ثم نصمم القطاعات بالترتيب كالأتى.

① نبدأ بتصميم القطاع الموجود على الكمره المؤثر عليه أكبر moment
و ذلك لتحديد عمق القطاع «t»

take $C_1 = 3.5$, $J = 0.78$ The sec. as R-Sec.

Get $d_\circ = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} \longrightarrow d_\circ \longrightarrow t_\circ = d_\circ + cover$

take $\boxed{t_1 = (1.1 \rightarrow 1.3)\, t_\circ}$

$\therefore$ $\boxed{\text{Check } \dfrac{P}{F_{cu}\, b\, t}}$

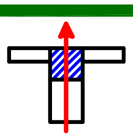
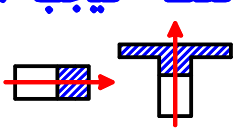
ⓐ IF $\dfrac{P_{U.L.}}{F_{cu}\, b\, t} \leqslant 0.04 \longrightarrow$ neglect $N_{U.L.}$

and Design the Sec. on B.M. only as Beams.

$\therefore\ d = d_1 = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}}$ take $C_1 = 3.5$, $J = 0.78$ (R-Sec.)

take $C_1 = 6.0$, $J = 0.826$ (T-Sec. , L-Sec.)

ملحوظه هامه :


في بدايه التصميم نعمل تصميم على M, P على أن القطاع R-sec.
و لكن اذا أهملنا ال P فنعمل تصميم على M فقط فيجب مراعاه
اذا كان القطاع R-sec. or T-sec.


ⓑ IF $\dfrac{P}{F_{cu}\, b\, t} > 0.04 \longrightarrow$ Don't neglect $P_{U.L.}$

and the sec. designed as R-sec.

Get A_s From e_s IF Ten. Failure $\longrightarrow A_s' =$ Stirrup Hangers

Get A_s From I.D. IF Comp. Failure $\longrightarrow A_s' = A_s$

② نصمم باقي قطاعات الكمره على نفس ال depth لـ Sec.①

③ ثم نصمم القطاع الموجود على العمود حيث يؤثر M & P كبيره

take $C_1 = 3.5$, $J = 0.78$

Get $d_o = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \rightarrow d_o \rightarrow t_o = d_o + cover$

take $t_2 = (1.1 \rightarrow 1.3)\, t_o$

Get A_s From e_s IF Ten. Failure $\rightarrow A_s` =$ Stirrup Hangers $\simeq 0.4\, A_s$

Get A_s From I.D. IF Comp. Failure $\rightarrow A_s` = A_s$

ملحوظه هامه

IF $t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)}$

و هذا فى ال Frames فقط و ليس فى ال Girders

أى ليس مع أعمده ال Link members

ملحوظه هامه

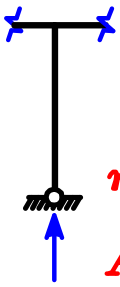
اذا كان العمود ليس Link member و لكن يوجد عليه Normal فقط

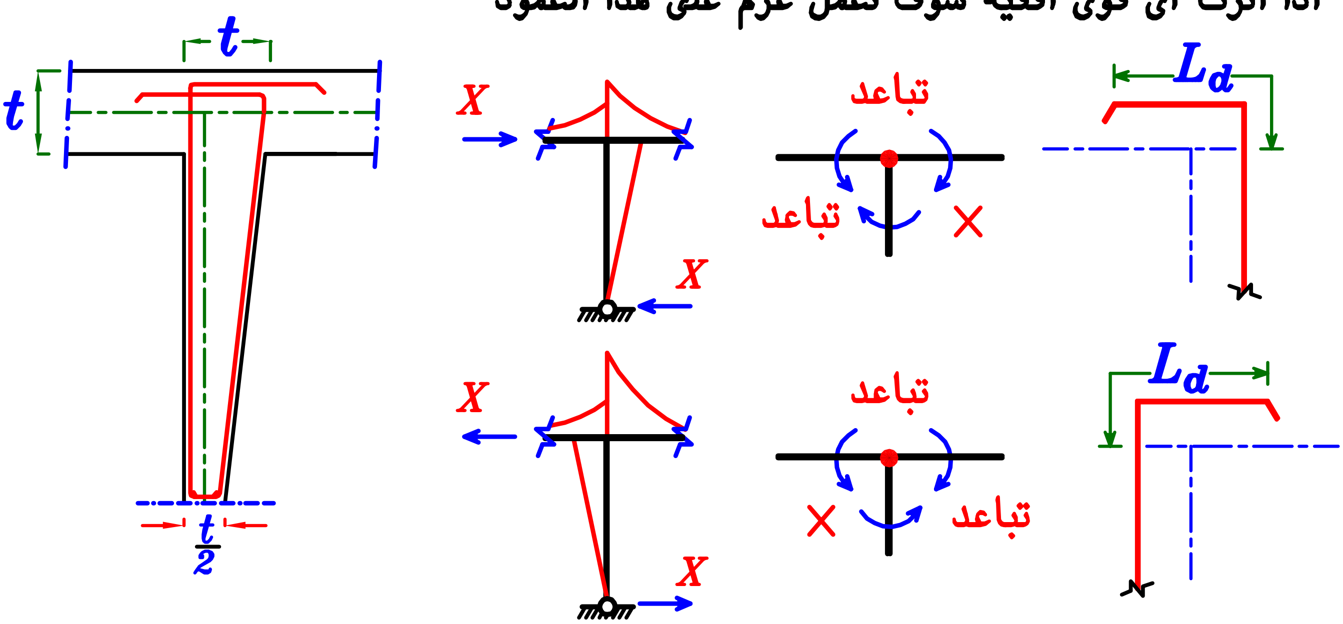
سنأخذ تخانته نفس تخانه الكمره و سنعمل على تصميم القطاع على Normal فقط

و سوف يكن التسليح أقل من ال minimum لذا سنأخذه يساوى ال minimum

$A_{s_{total}} = A_{s_{min.}} = \frac{0.8}{100} * b * t \xrightarrow{Take} A_s = A_s` = \frac{A_{s_{min.}}}{2}$

اذا أثرت أى قوى أفقيه سوف تعمل عزم على هذا العمود



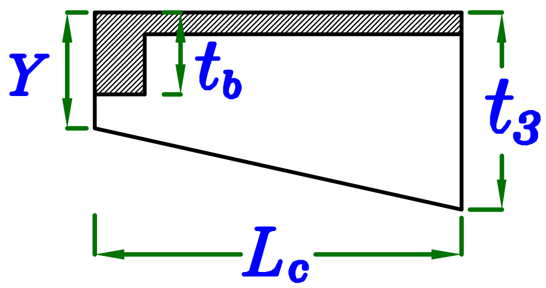


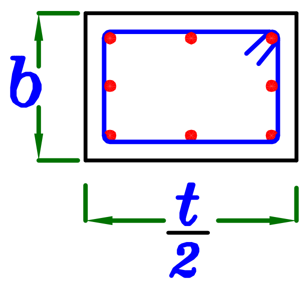
④ ثم نصمم قطاع ال **Cantilever** (**Sec.④**)

و يكون له عمق مختلف عن باقى ال **Frame**

$$d_3 = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \qquad C_1 = 3.5 \ , \ J = 0.78 \quad , \quad t_3 = d_3 + Cover$$

$$Y = \left\{ \begin{array}{l} \dfrac{t_3}{2} \\ t_b = \dfrac{Spacing}{12} \\ t_3 - \dfrac{L_c}{3} \end{array} \right\}$$ الأكبر





⑤ ثم نصمم قطاع ال **Link member** (**Sec.⑤**)

و عاده تؤخذ أبعاد هذا القطاع ($b * \frac{t}{2}$)

و يصمم كأنه عمود $P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$

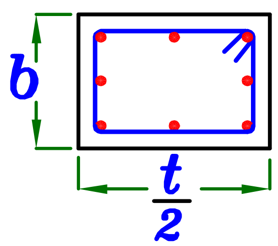
ملحوظه

اذا كان ال **Link member** عليه **Tension** و ليس **Compression**

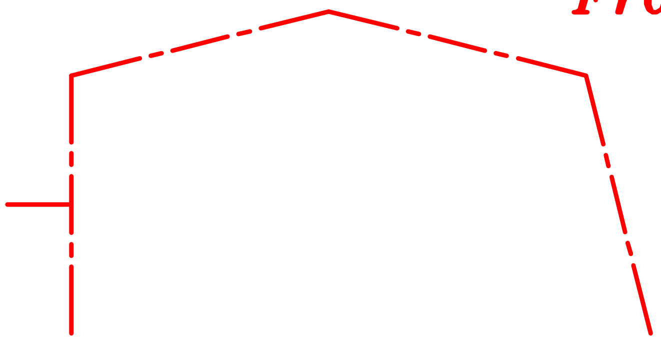
تؤخذ أبعاد هذا القطاع ($b * \frac{t}{2}$)

$$\boxed{A_s = \frac{T_{U.L.}}{F_y / \gamma_s}}$$

و يصمم كأنه **Tie**

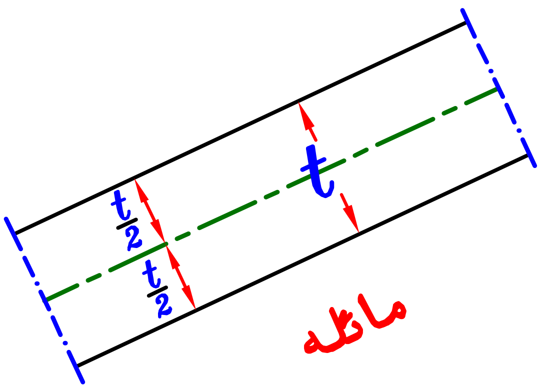


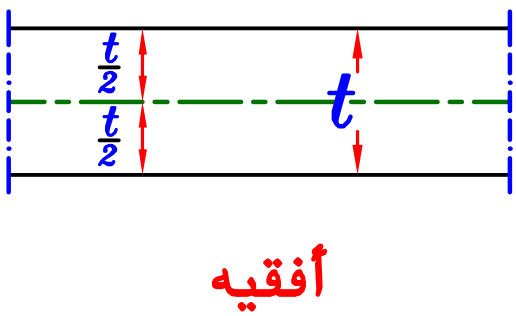
① نبدأ برسم الـ *C.L.* للـ *Frame*
بمقياس الرسم المطلوب



② نرسم عمق الـ *members* المختلفه بنفس مقياس الرسم
و ذلك حسب شكل كل *member* مثل :

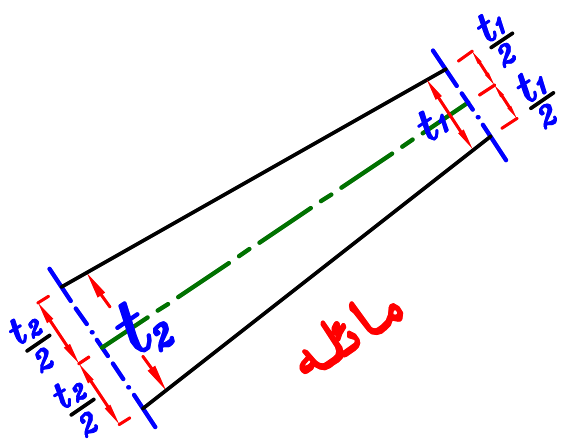
أ كمره لها عمق ثابت «t»

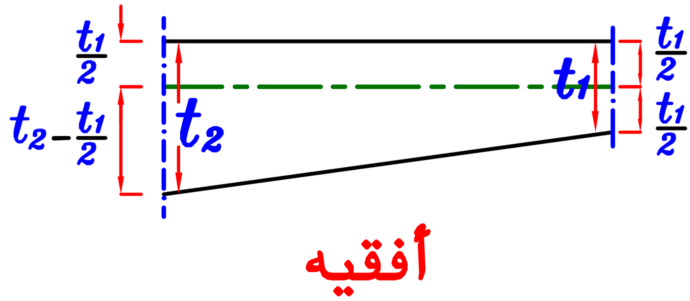




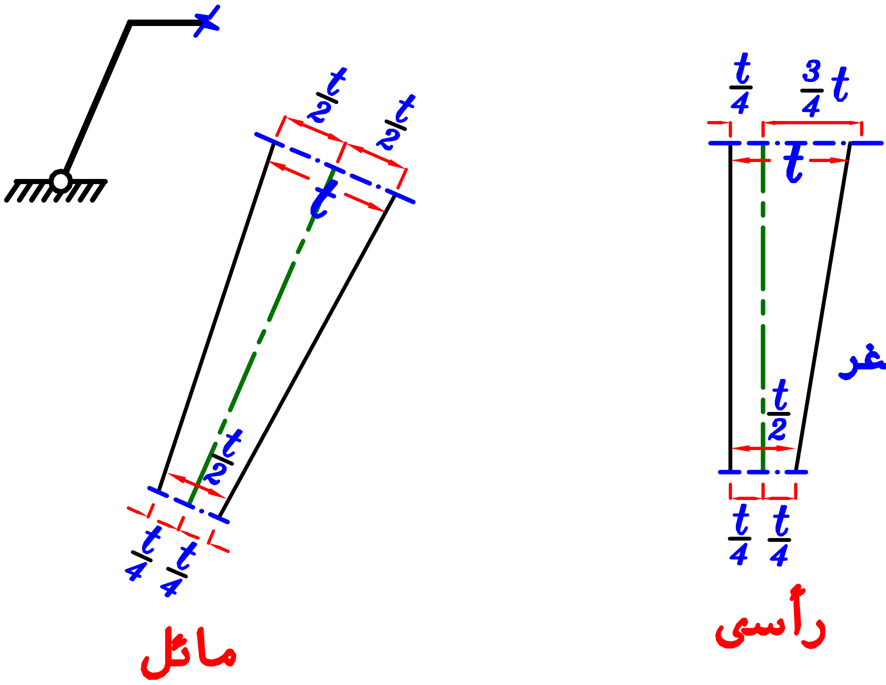
ب كمره لها عمقان مختلفان (t_2) & (t_1)

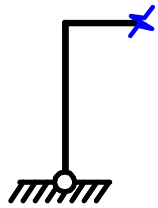
يكون الـ *C.L.* فى منتصف العمق الأصغر





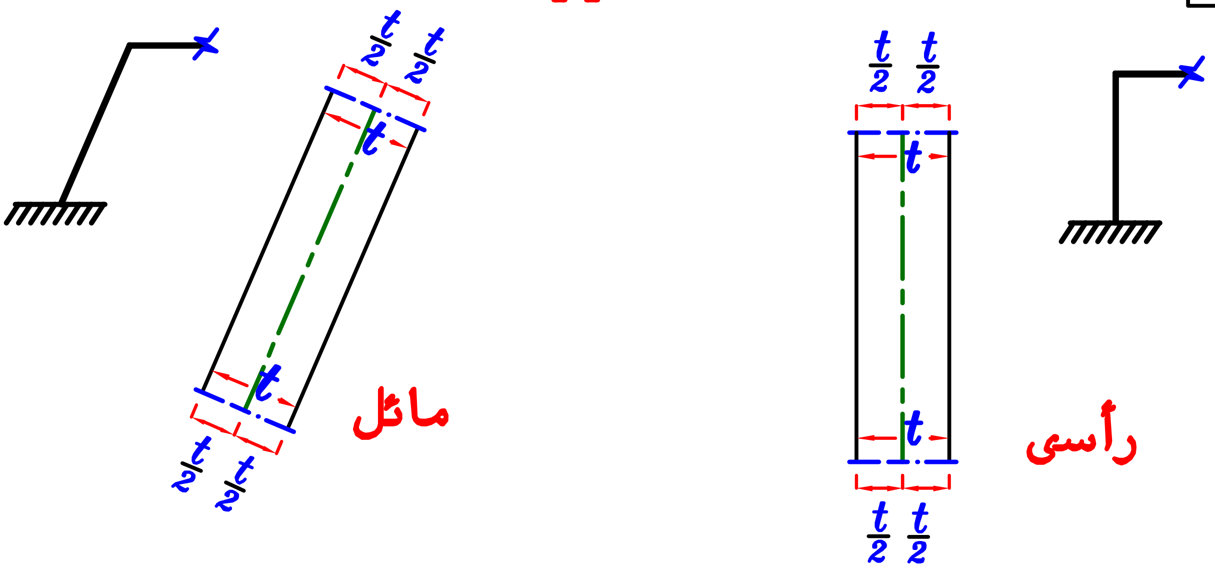
ج الأعمده التى أخرها Hinged support



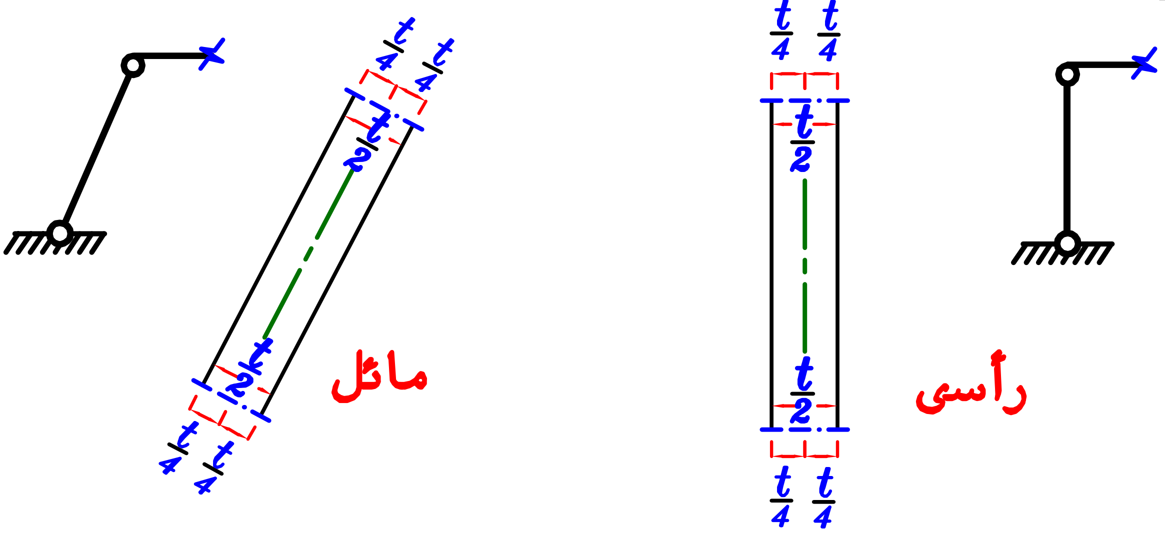


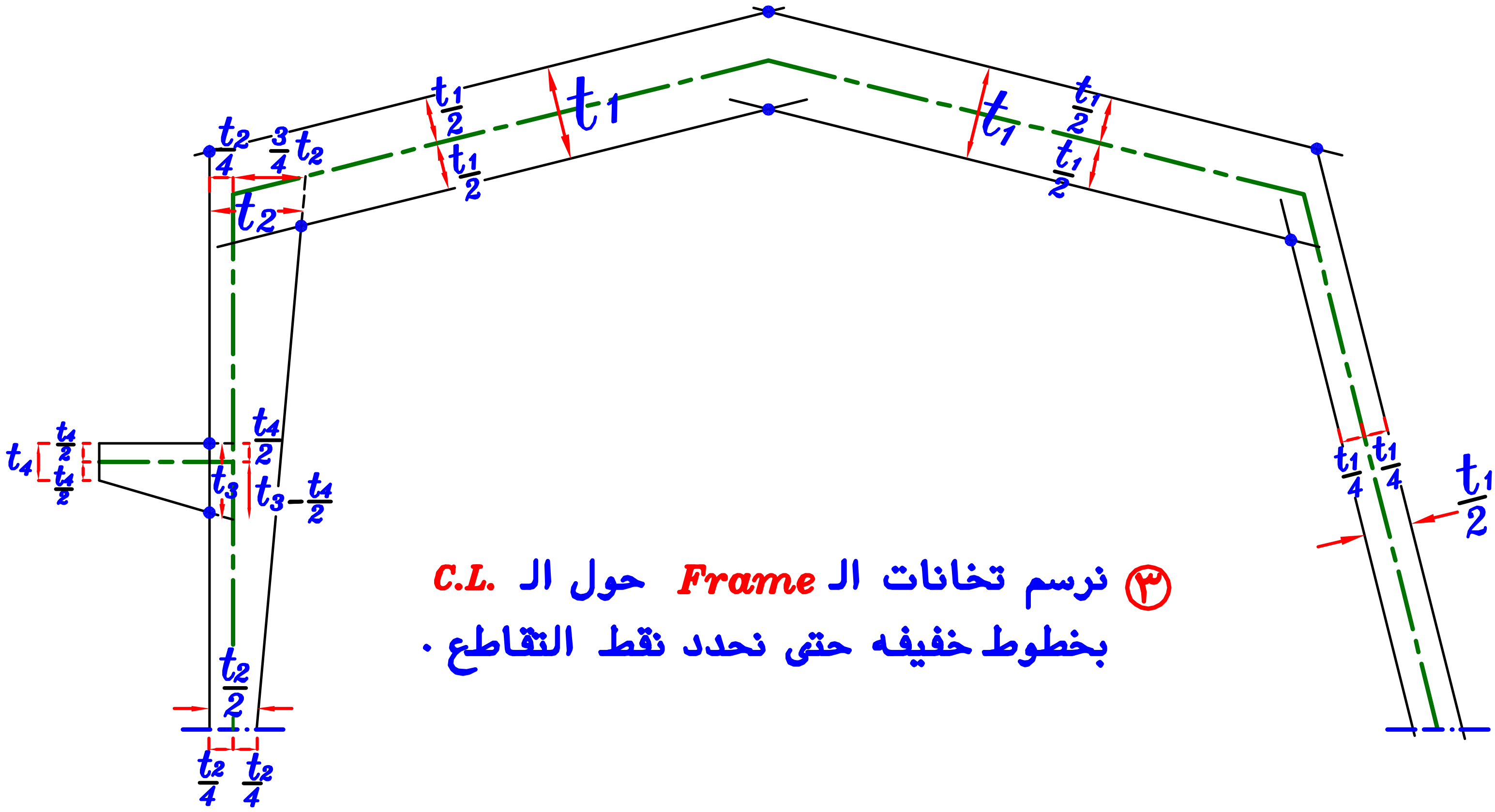
يكون ال C.L.
فى منتصف العمق الأصغر

د الأعمده التى أخرها Fixed support

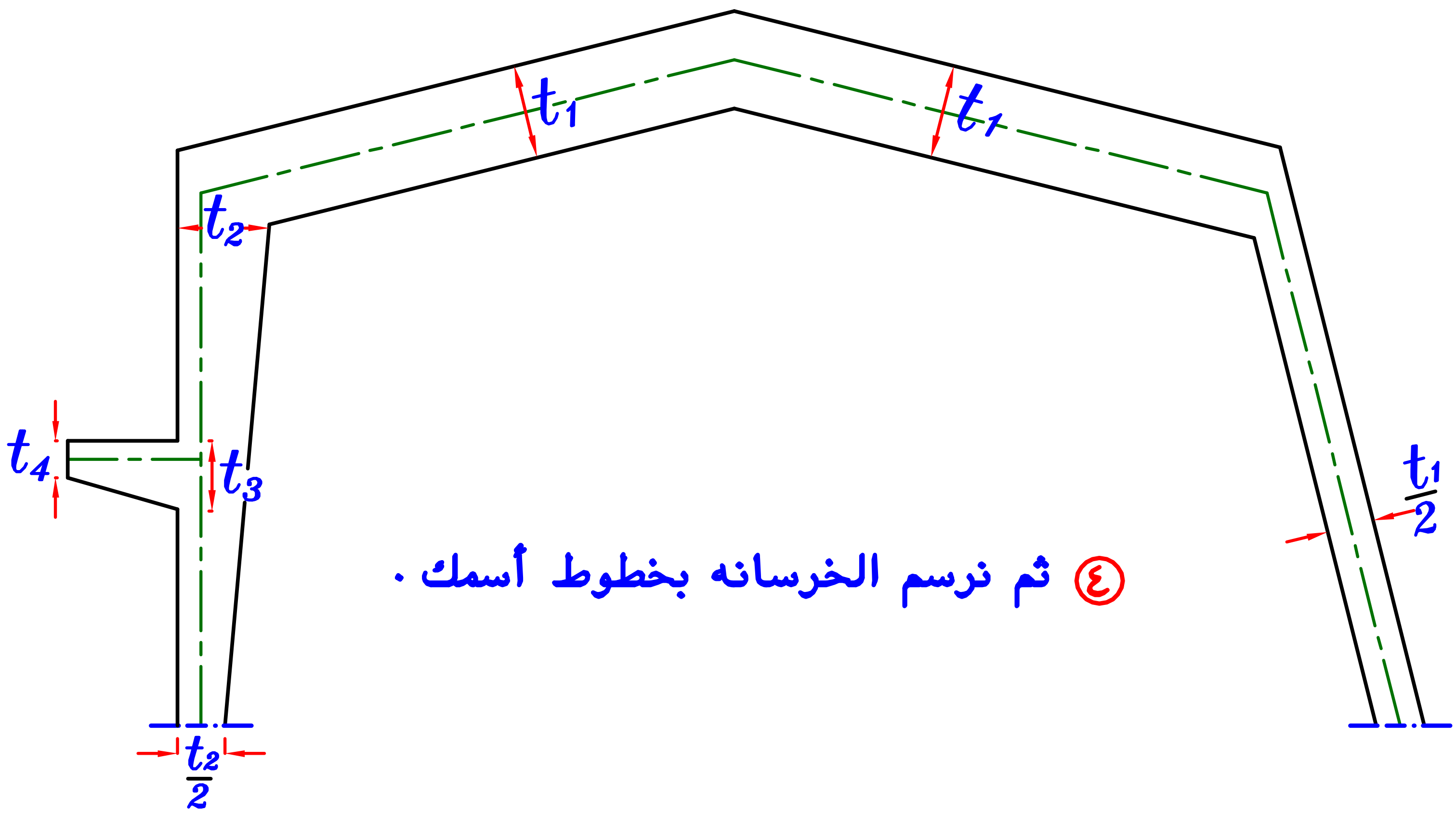


هـ الأعمده ال Link member.





③ نرسم تخانات الـ **Frame** حول الـ **C.L.**
بخطوط خفيفه حتى نحدد نقط التقاطع .

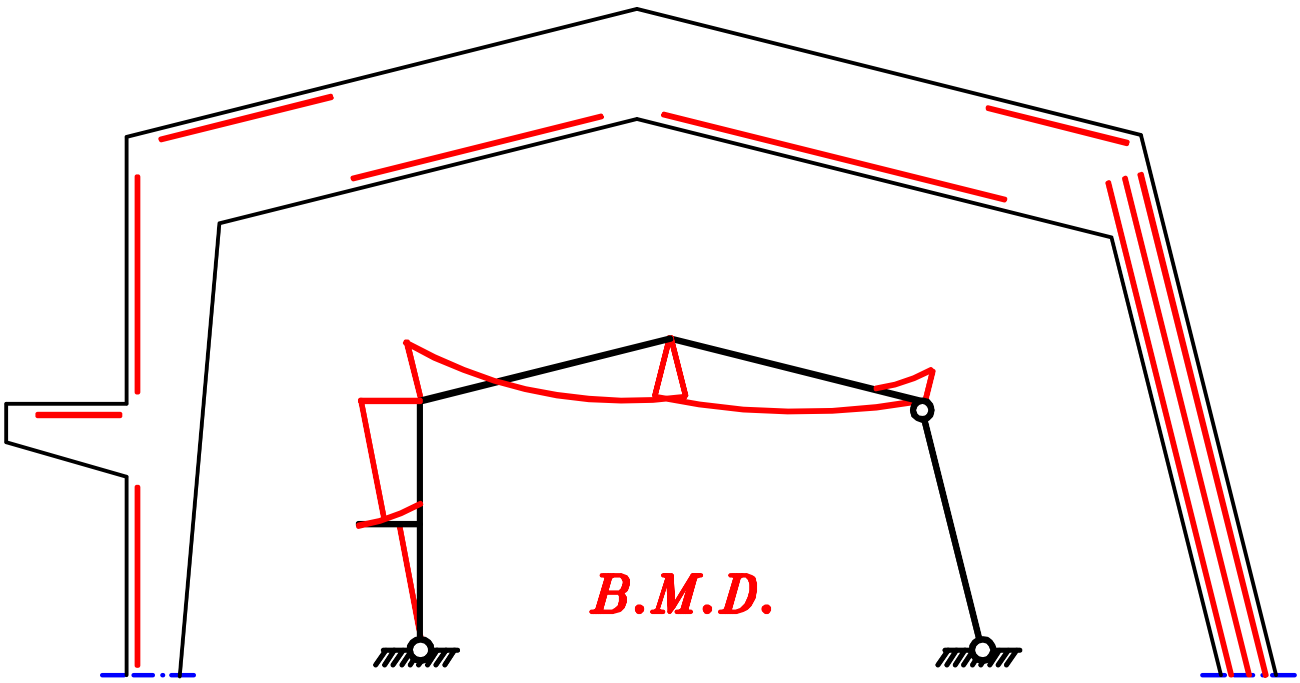


④ ثم نرسم الخرسانه بخطوط أسمك .

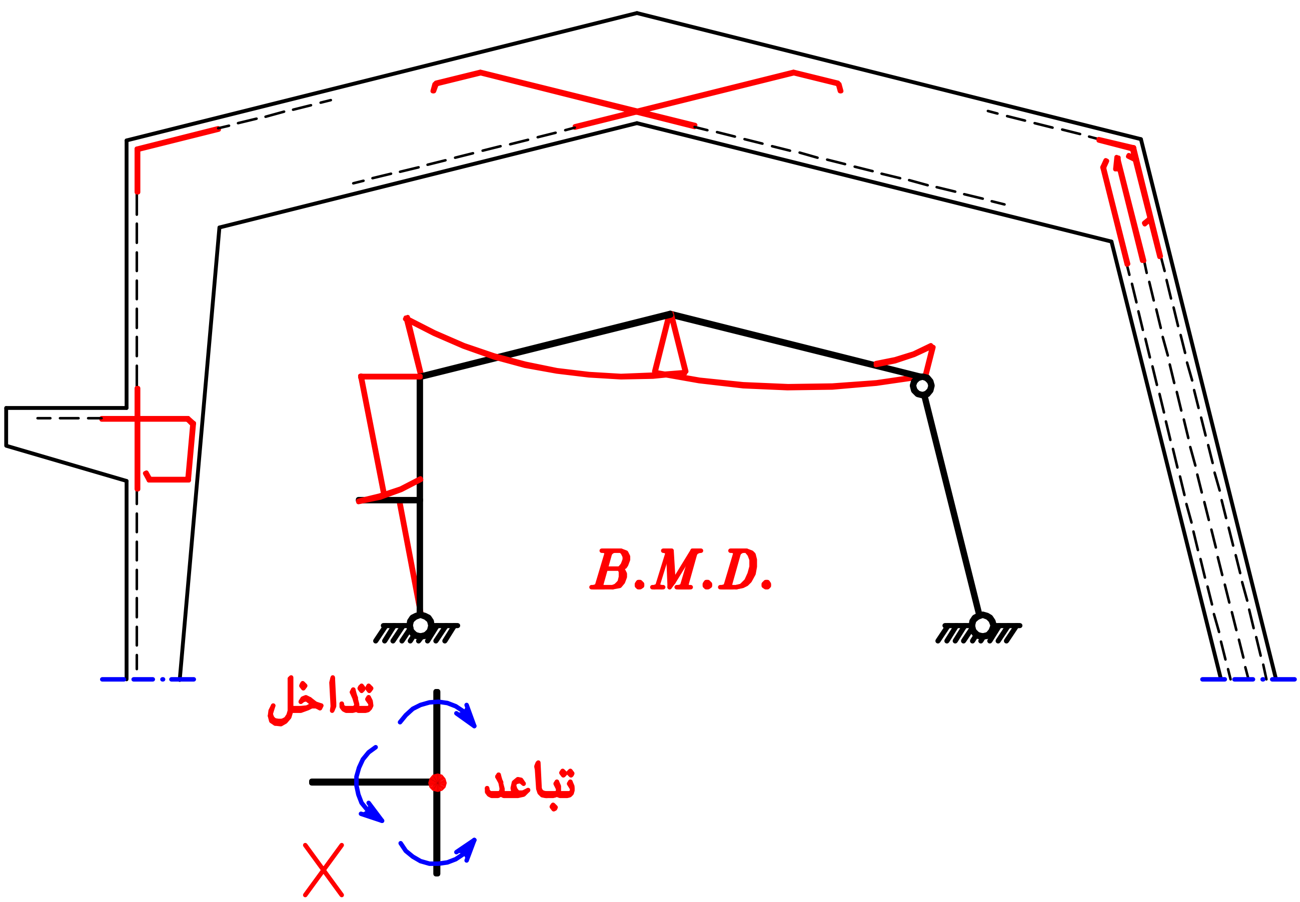
Steps to Draw Frame Reinforcement.

مراحل رسم التسليح لل *Frame* مع مراعاه الترتيب

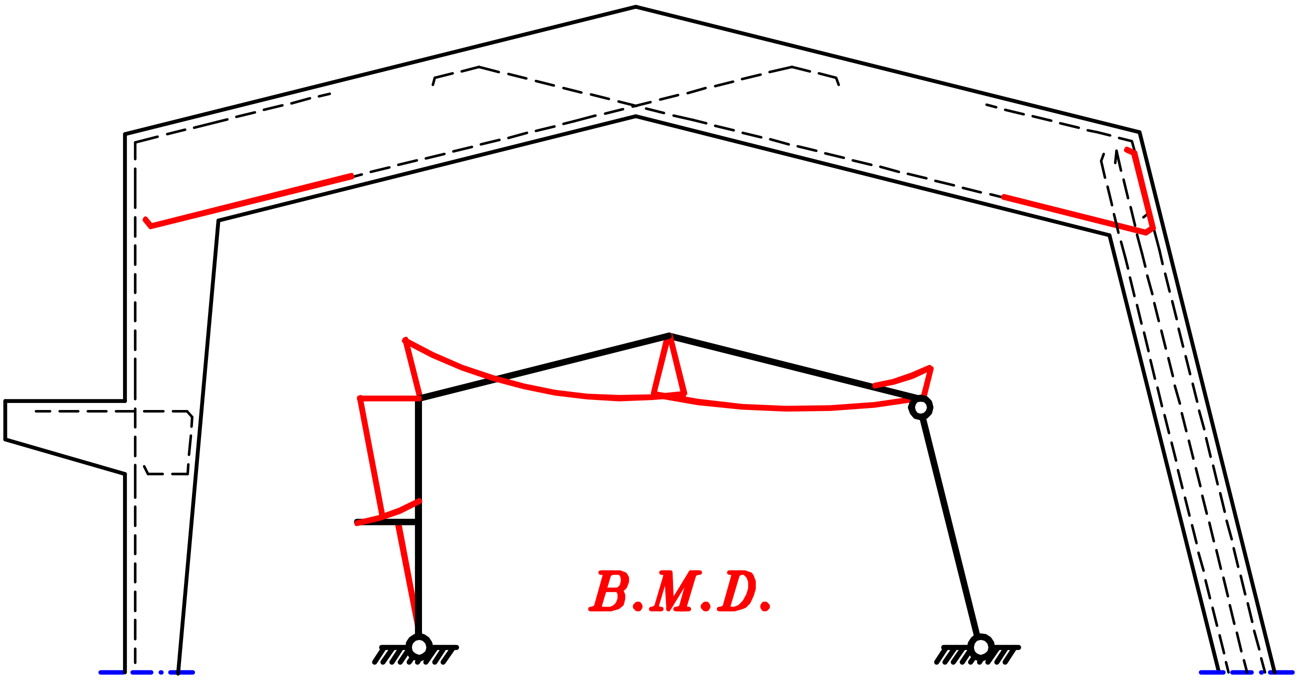
١- رسم مكان التسليح الرئيسى جهة ال *moment* .



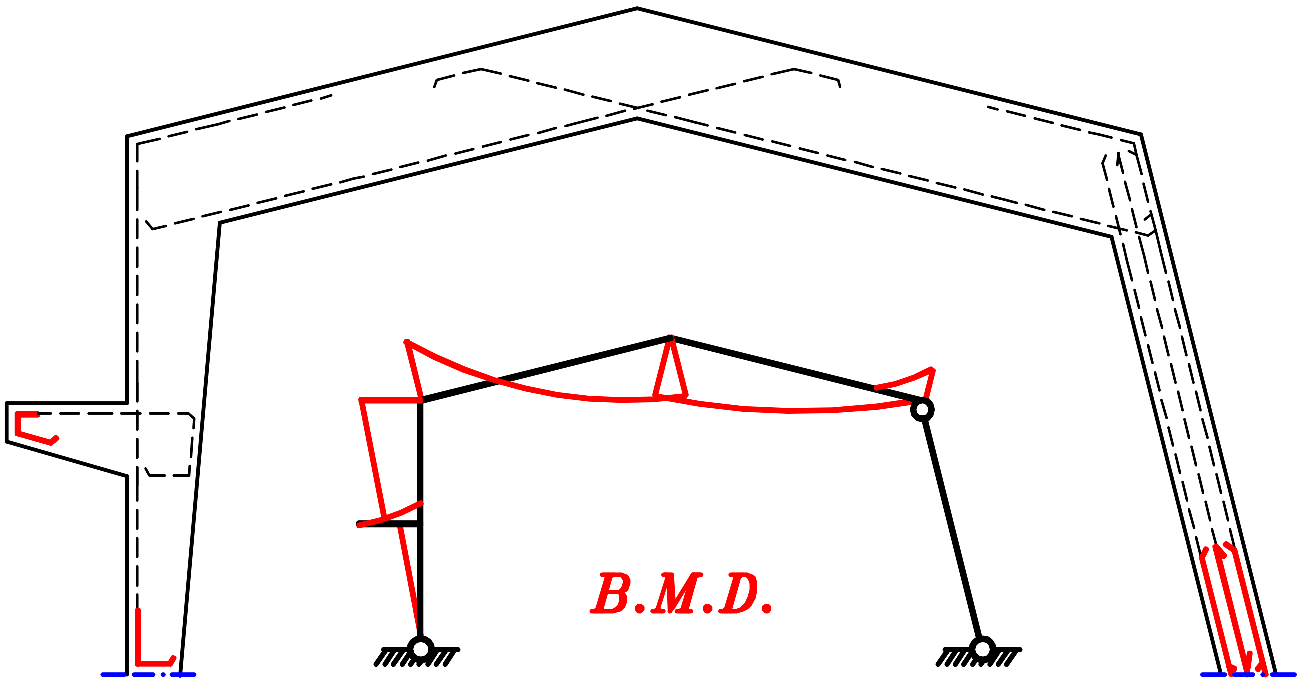
٢- تسليح ال *Joints* (عمل التباعد و التداخل) .



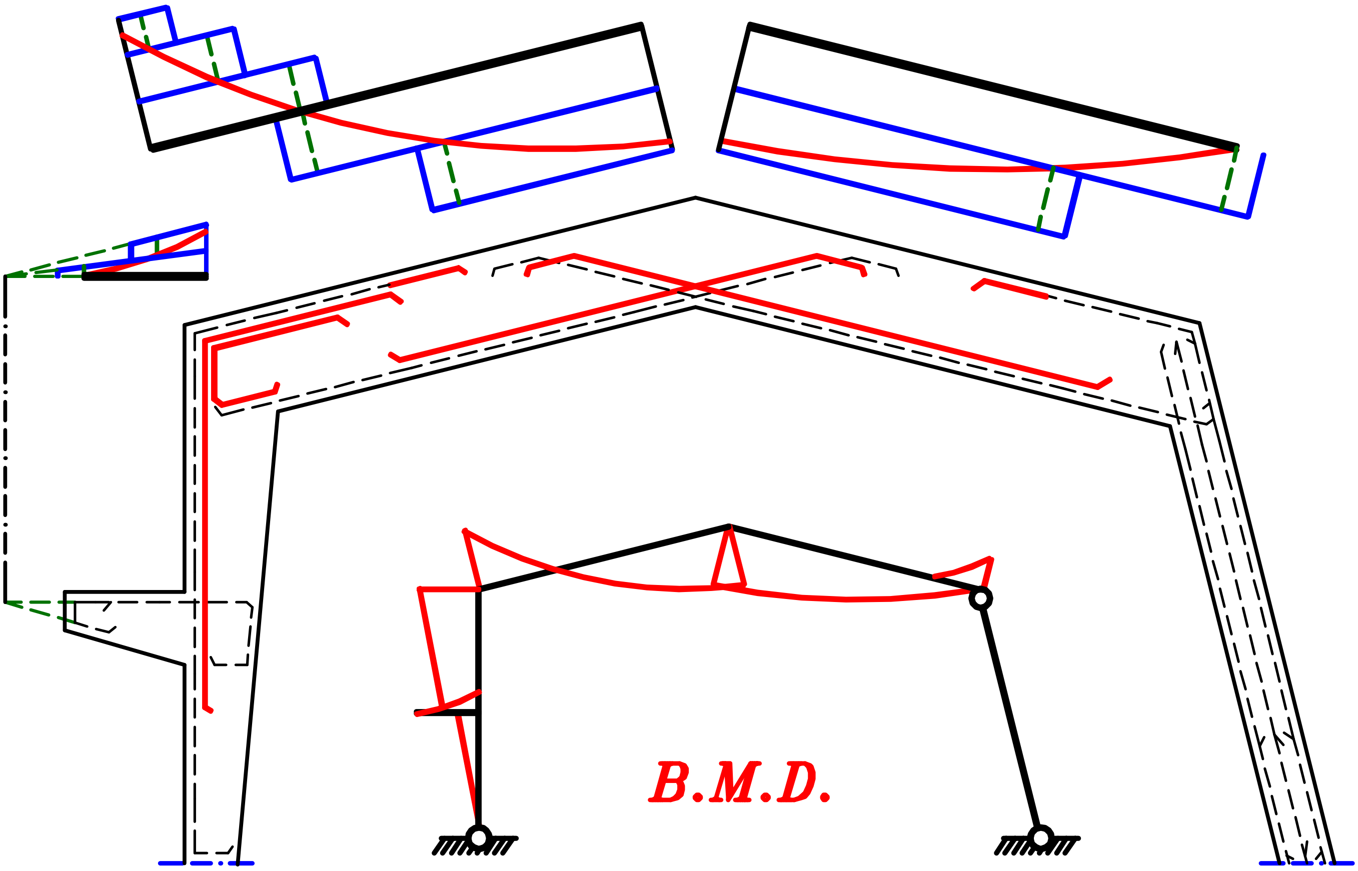
٣- الحديد السفلى يرسم من وش العمود الى وش العمود .



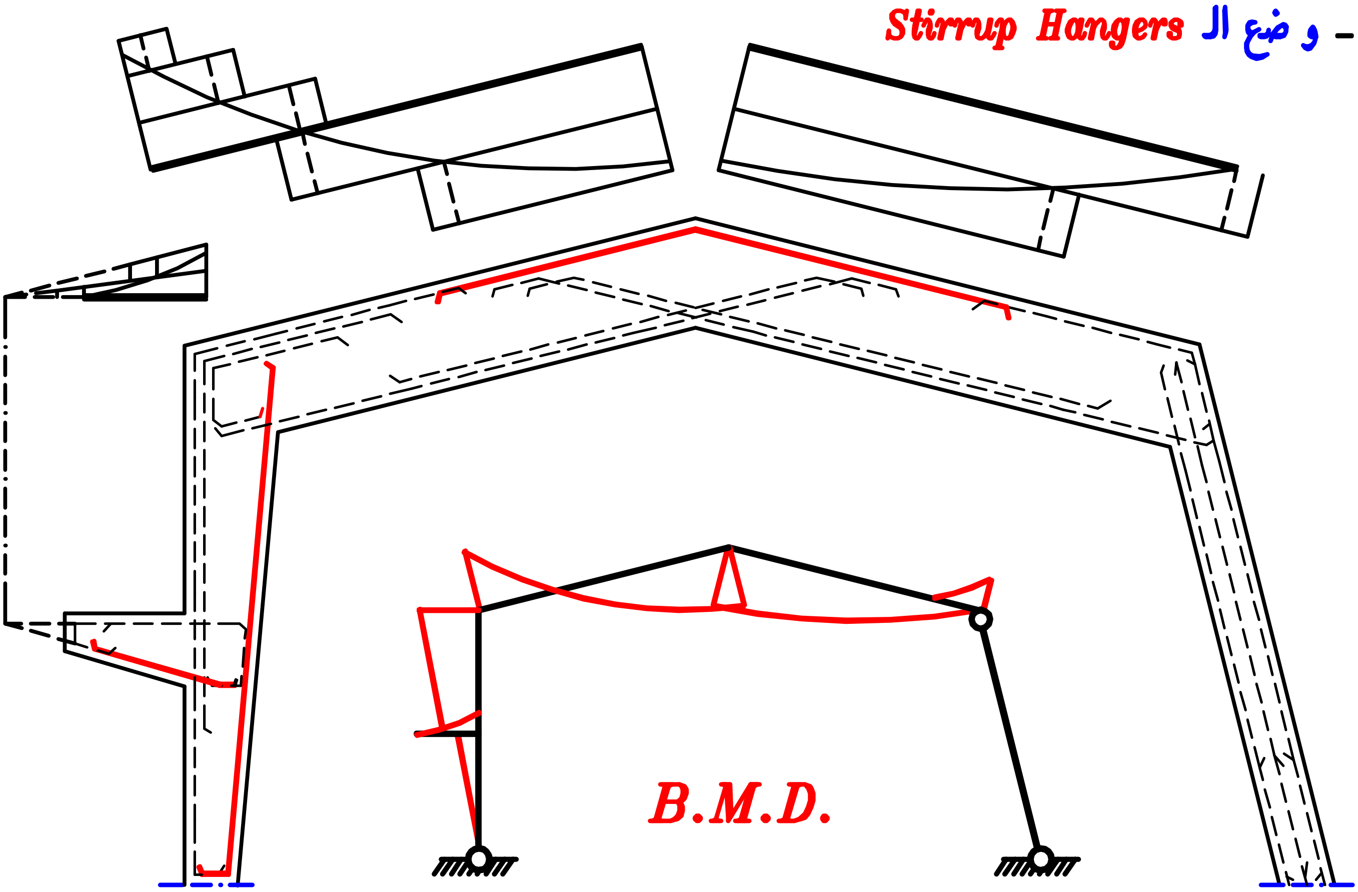
٤- رسم التسليح عند نهاية الاعمده (عمل أشاير أو لف الحديد فى العمود) و طرف ال Cantilever .



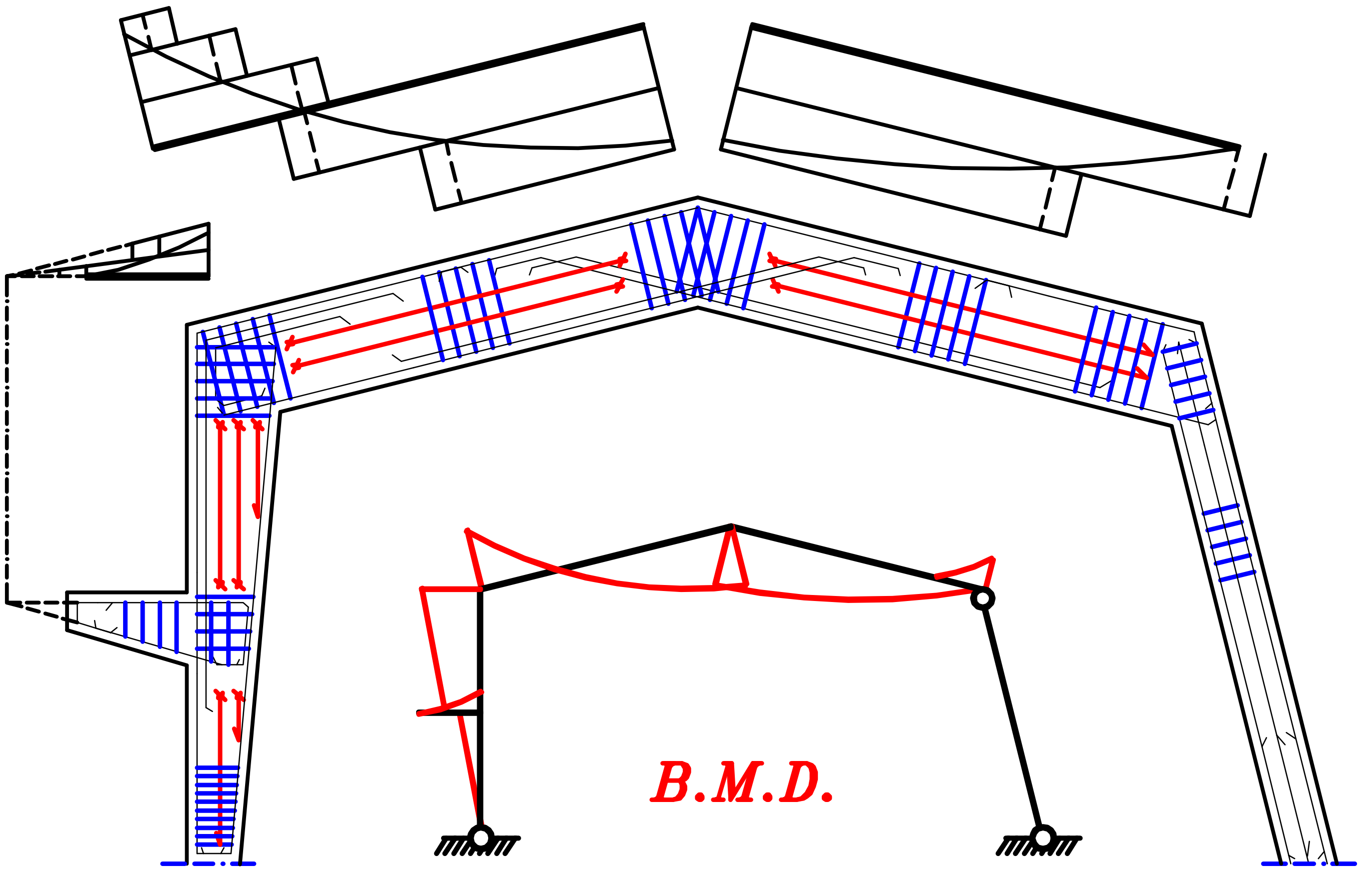
٥- رسم البلوكات مع مراعاه عدد الاسياخ و عدد الصفوف .



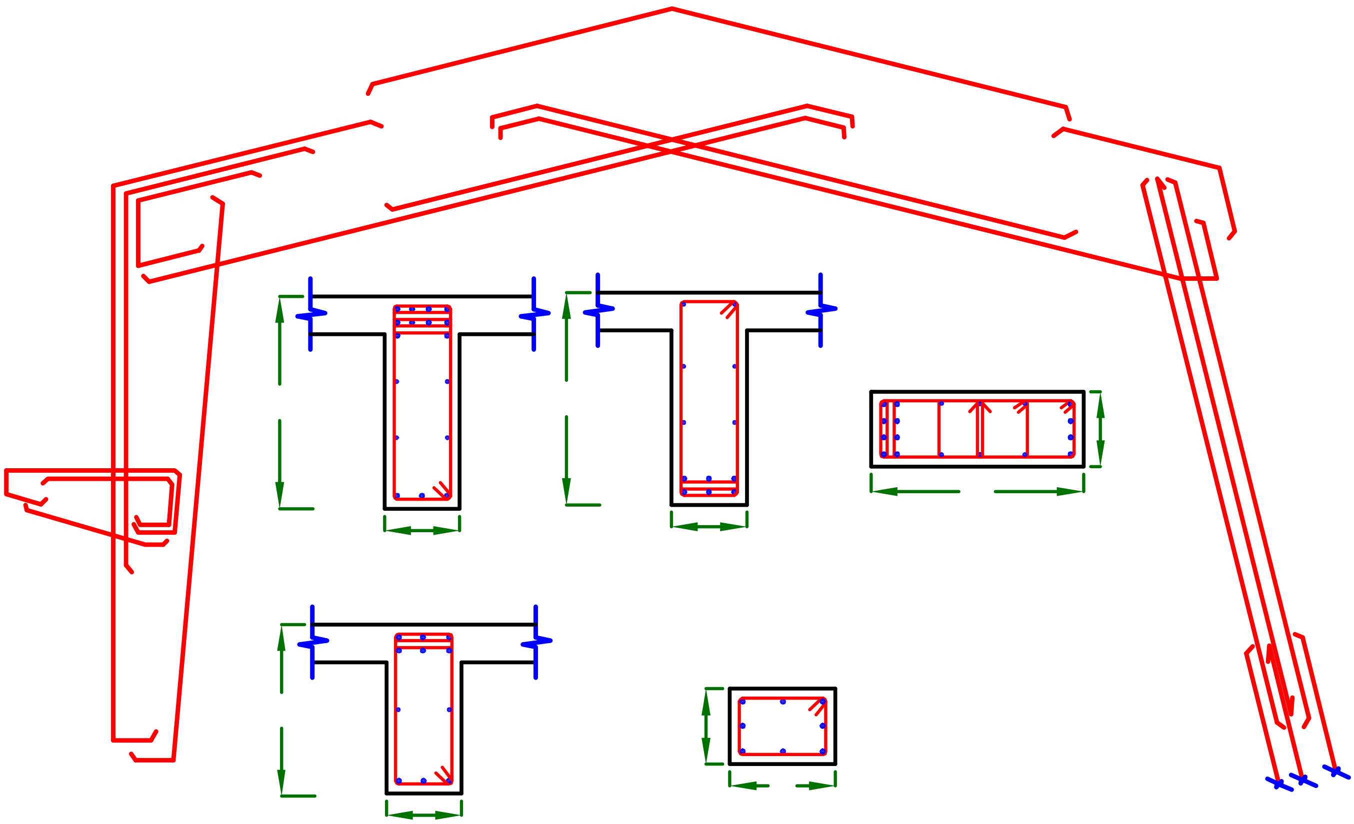
٦- و ضع ال Stirrup Hangers

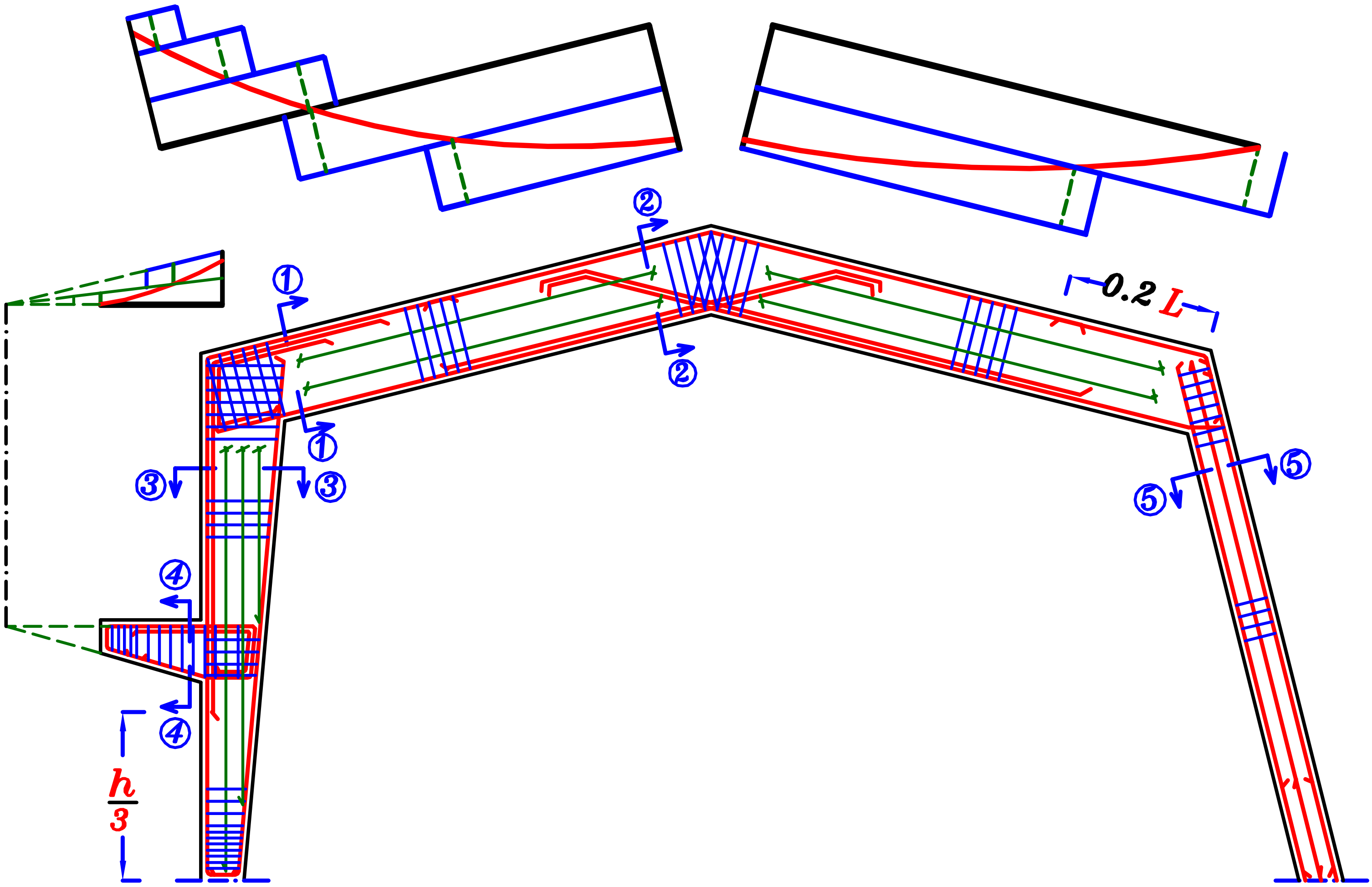


٧ـ وضع الكانات و ال Shrinkage bars و ال Buckling bars

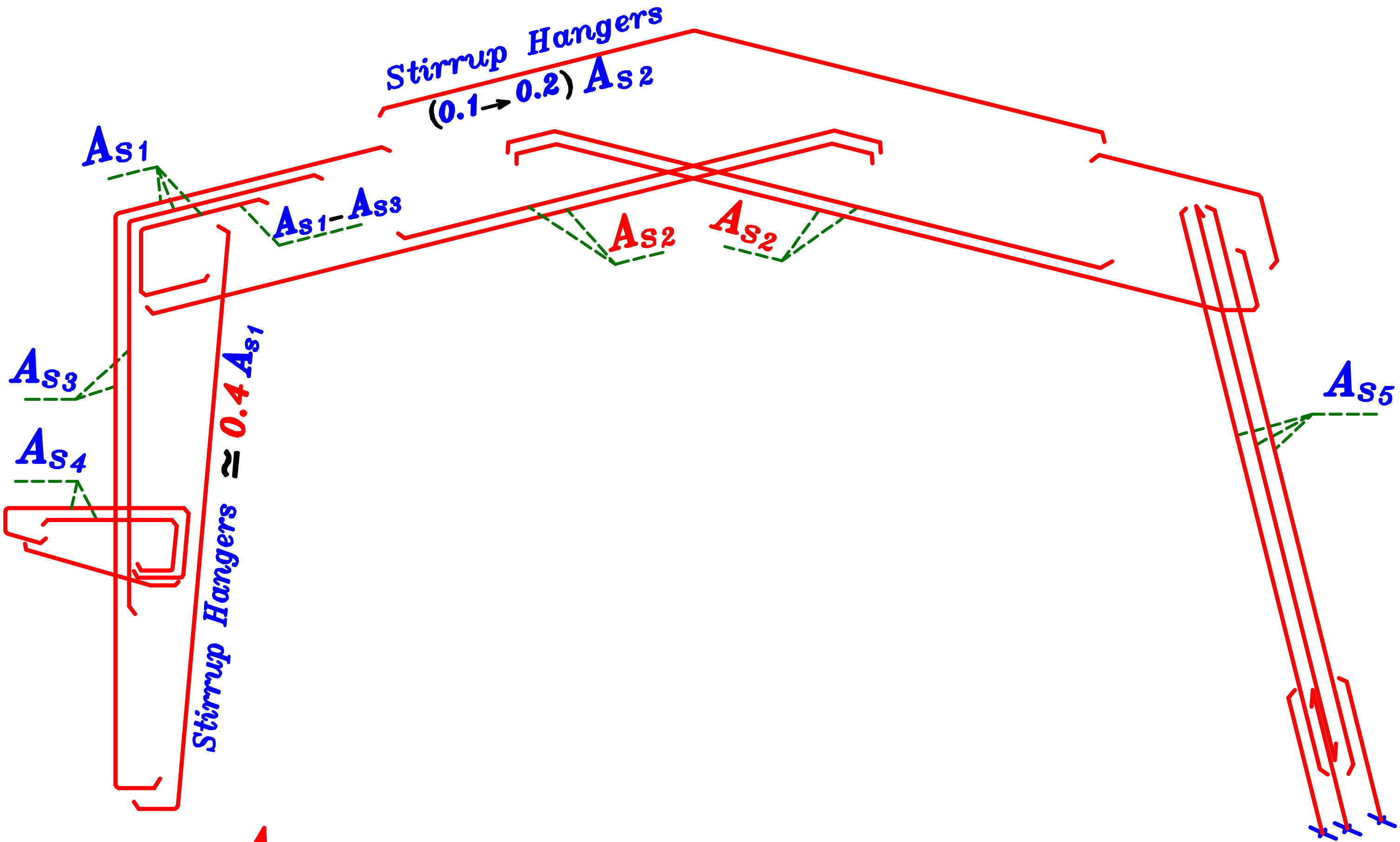


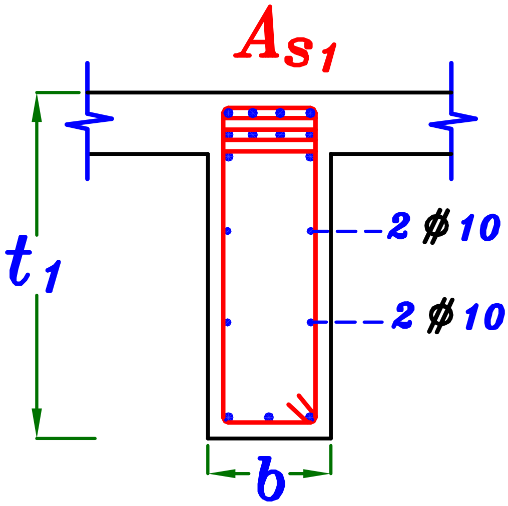
٨ـ رسم التفريد و ال Sections



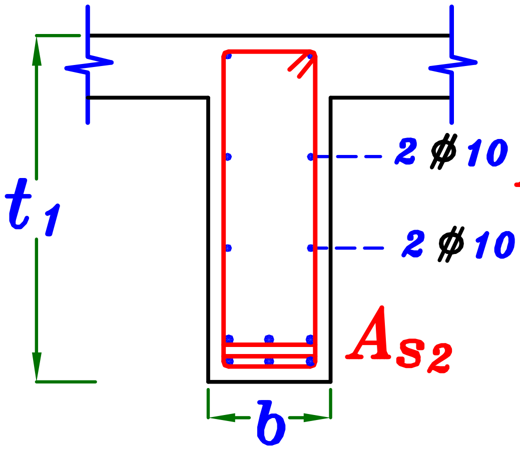


①
②
③
④
⑤
0.2 L
h
3

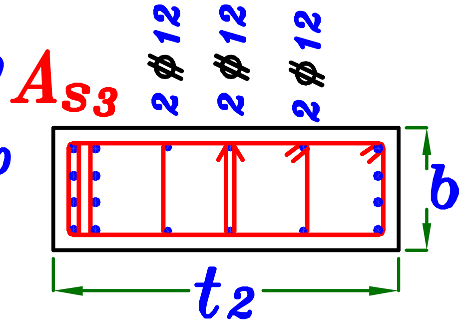




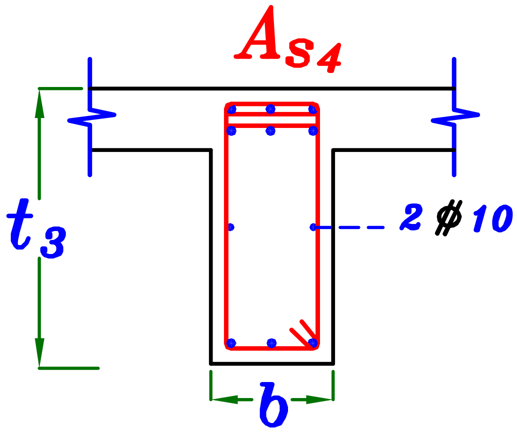
Sec. (1-1)



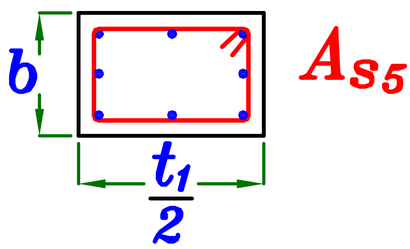
Sec. (2-2)



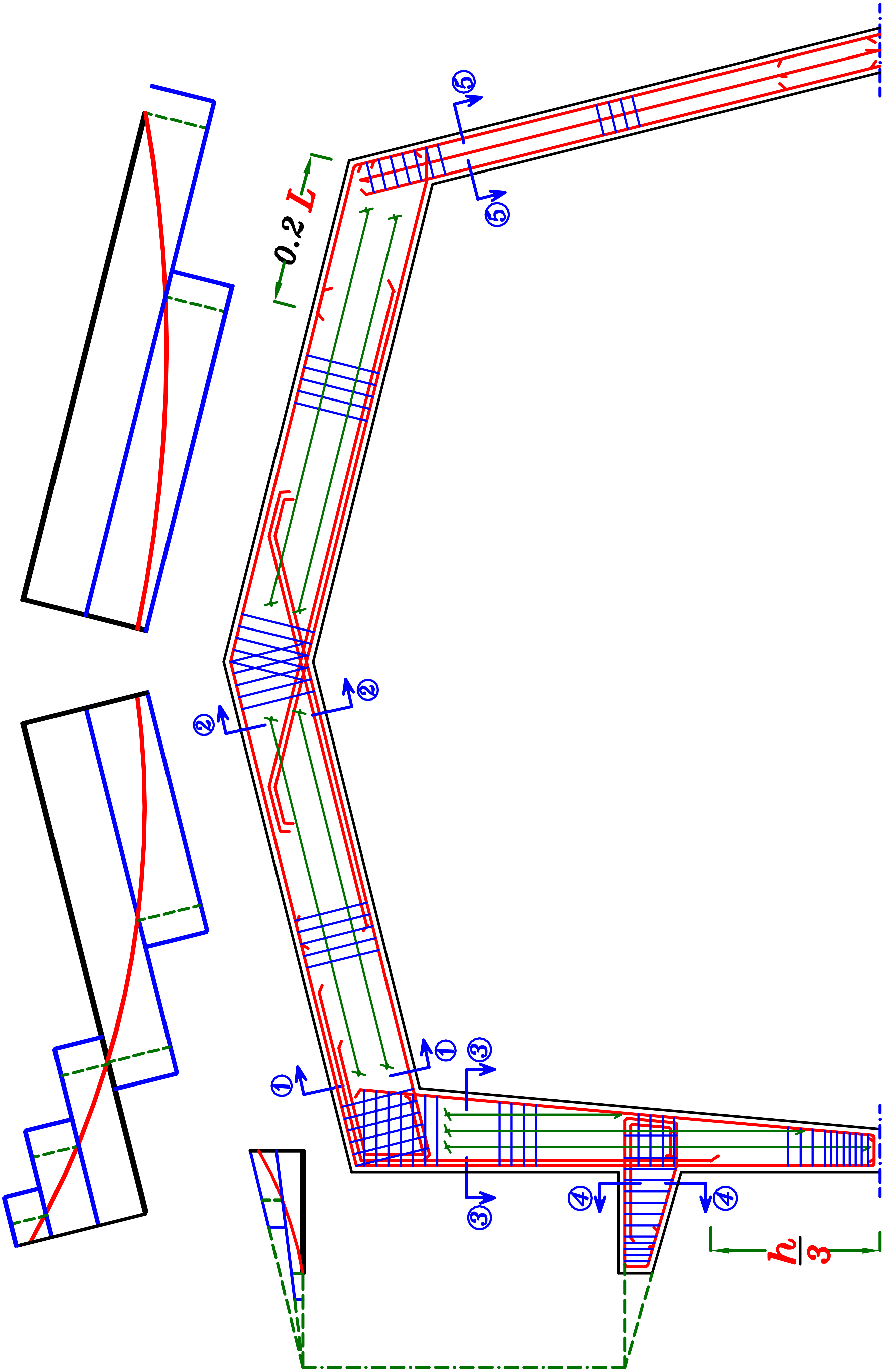
Sec. (3-3)

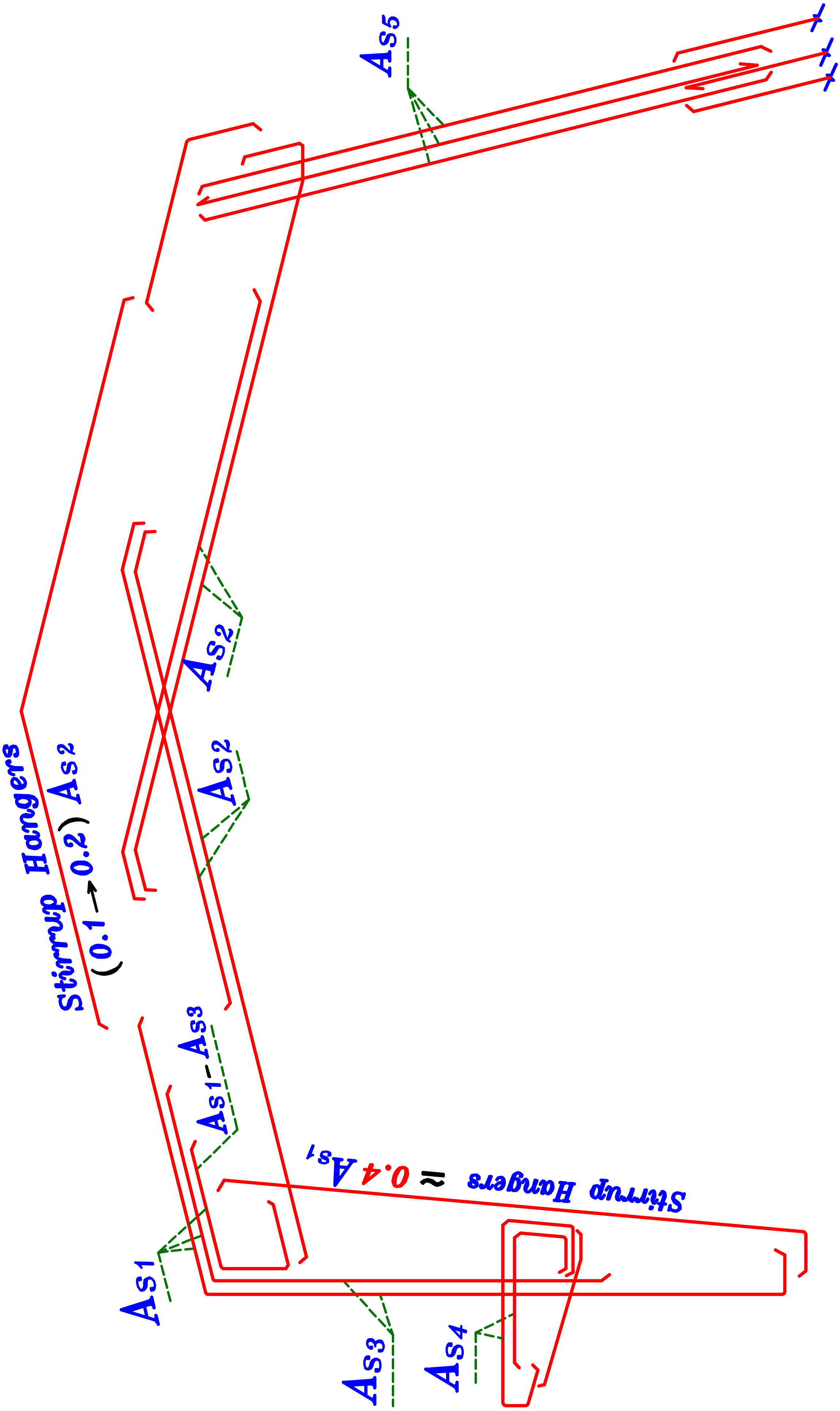


Sec. (4-4)



Sec. (5-5)


0.2 L
①
①
②
②
③
③
④
④
⑤
⑤
h/3

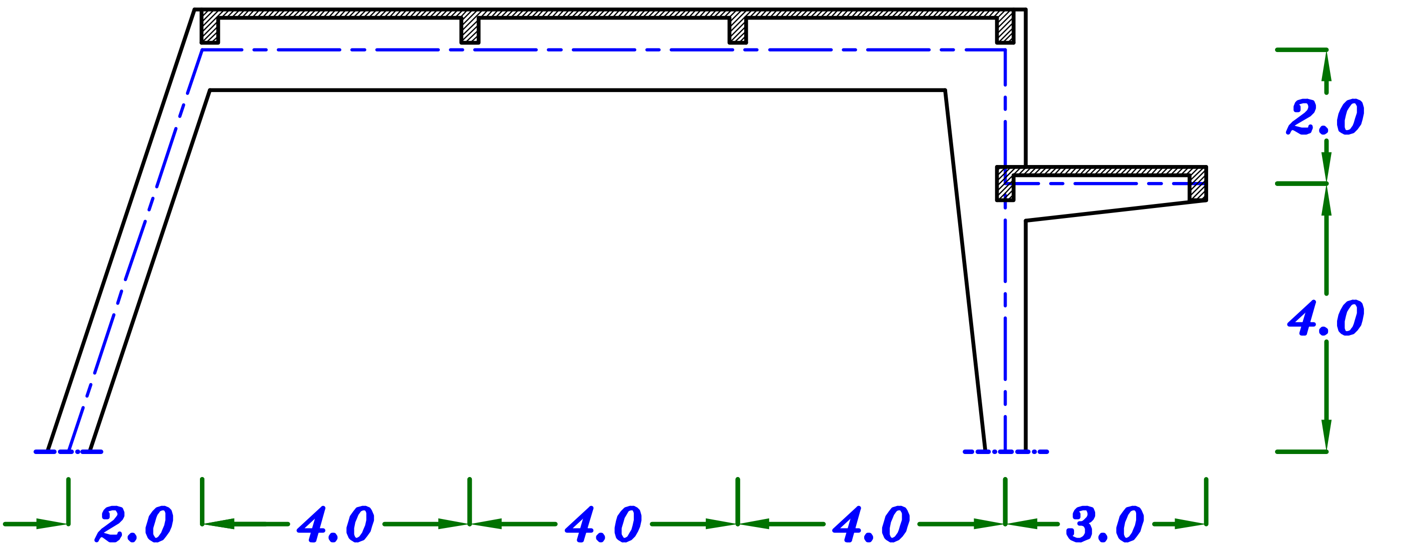

As5
As2
Stirrup Hangers (0.1→0.2) As2
As2
As1 - As3
Stirrup Hangers ≈ 0.4 As1
As1
As3
As4

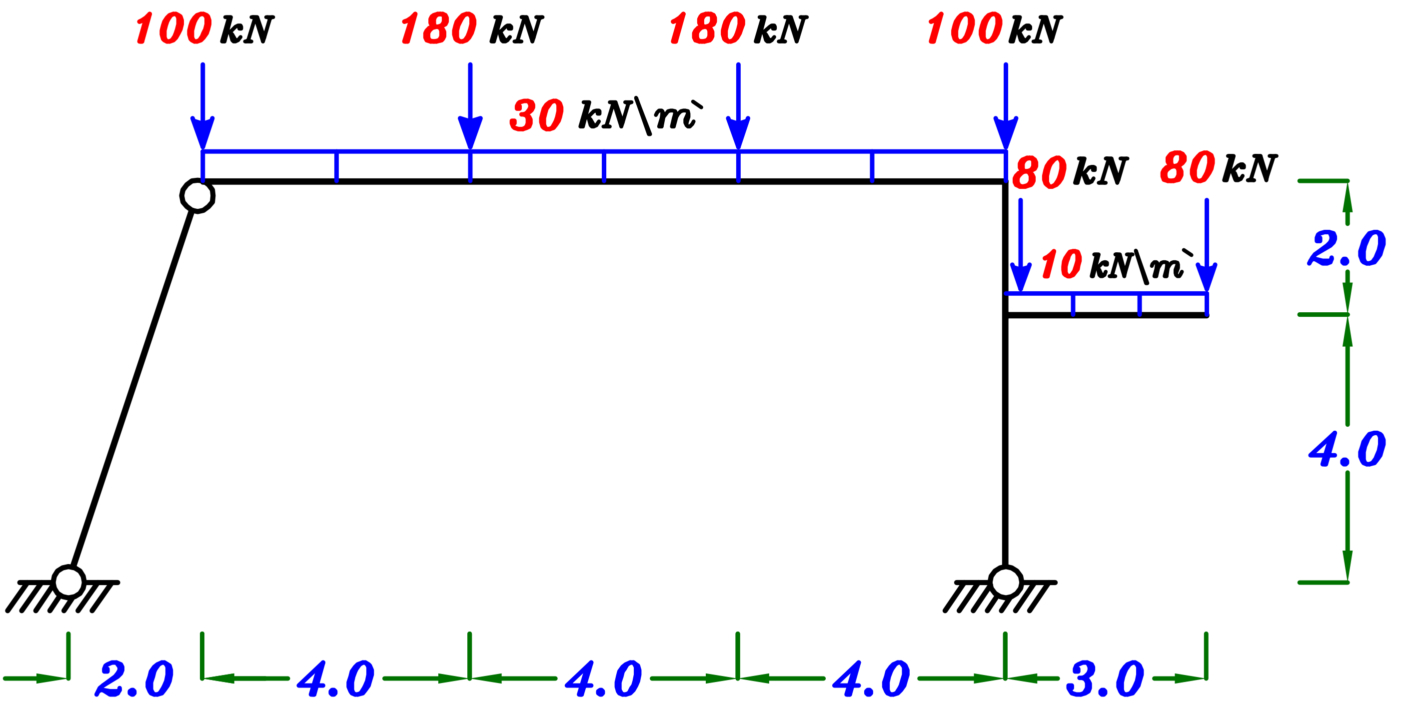
Examples on Frames.

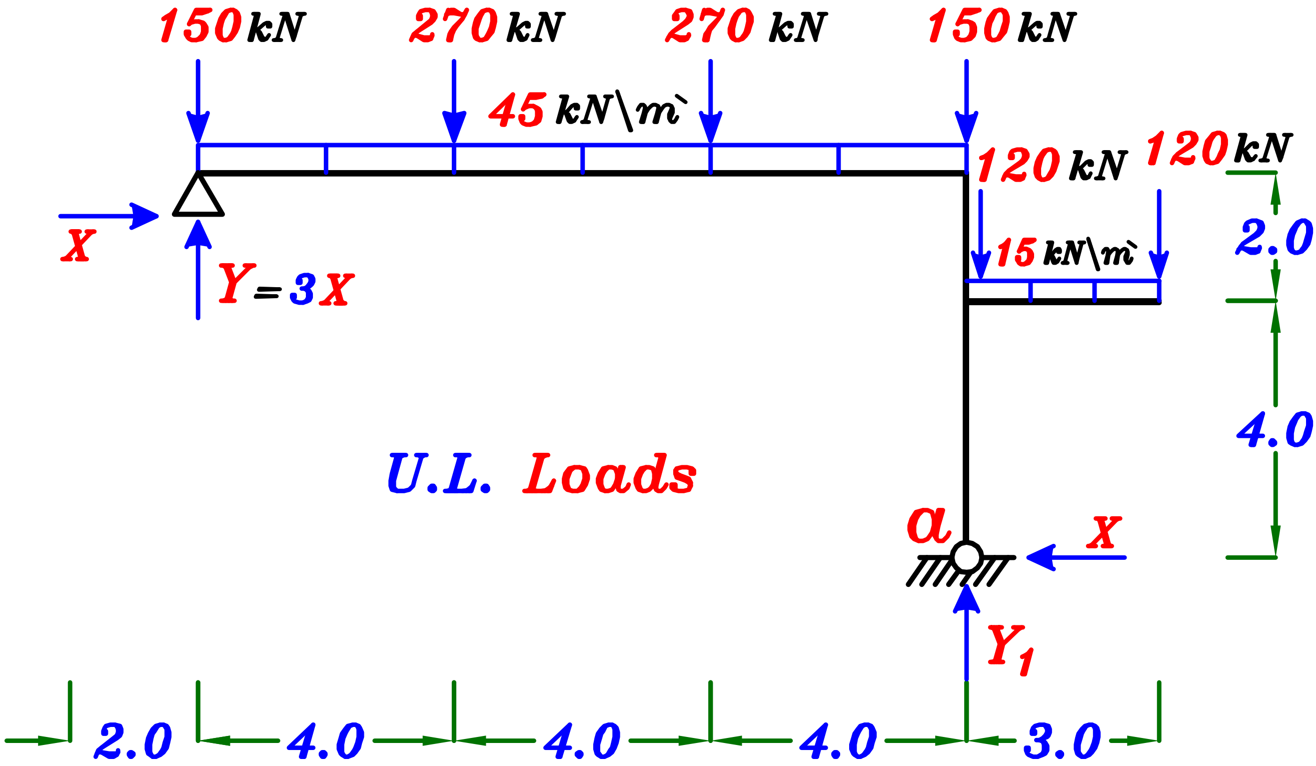
For the shown reinforcement concrete Frame of spacing 6.0 m and subjected to the given working loads.
The Following is required :

1- Draw the internal Force diagrams (B.M. , S.F. and N.F.) For the Frame.
2- Design the critical sections of the Frame For Bending.
3- Draw details of reinforcement of the Frame using moment of resistance diagram.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$, Spacing = 6.0 m
b (Beams) = 250 mm , b (Frame) = 350 mm , $t_s = 120$ mm



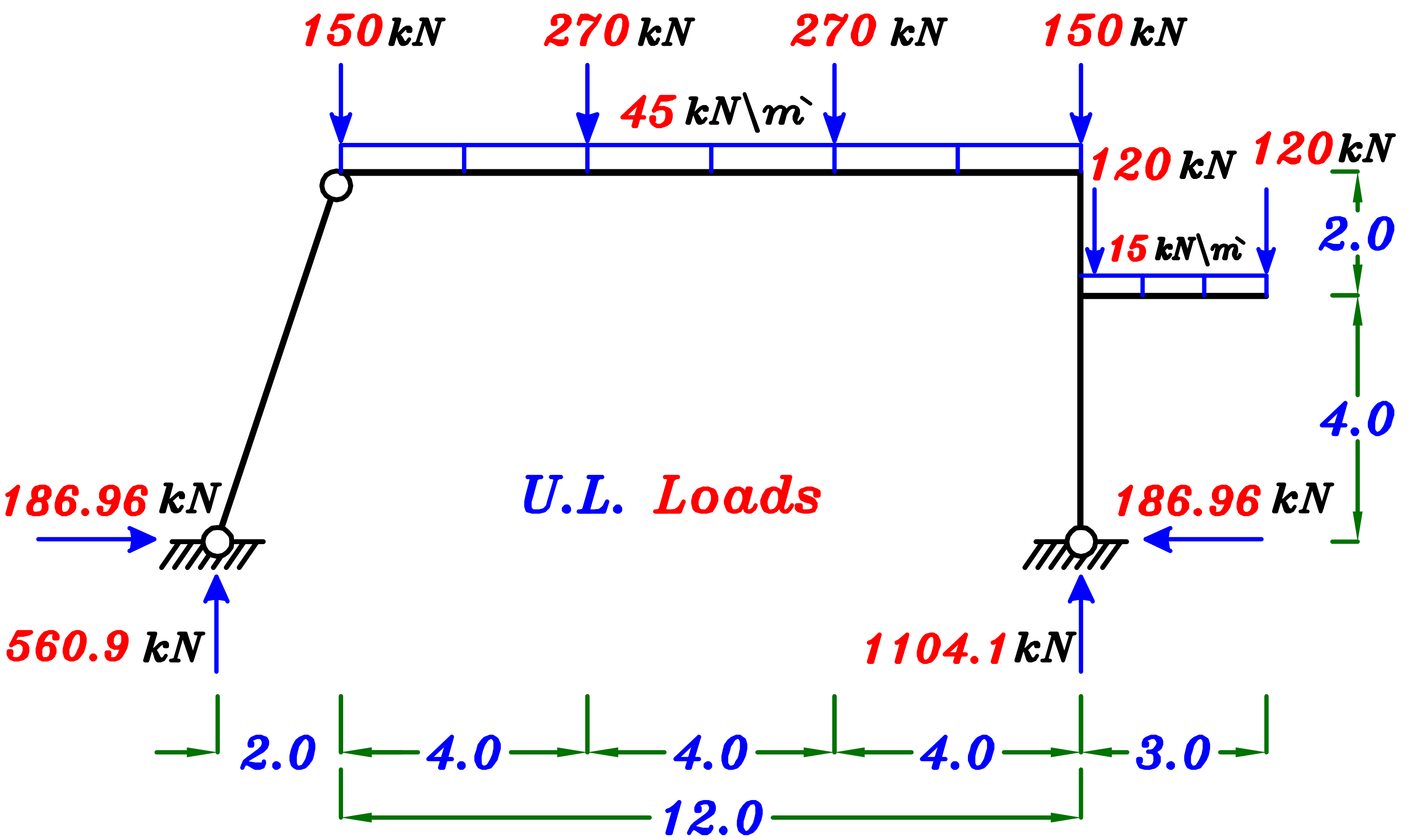


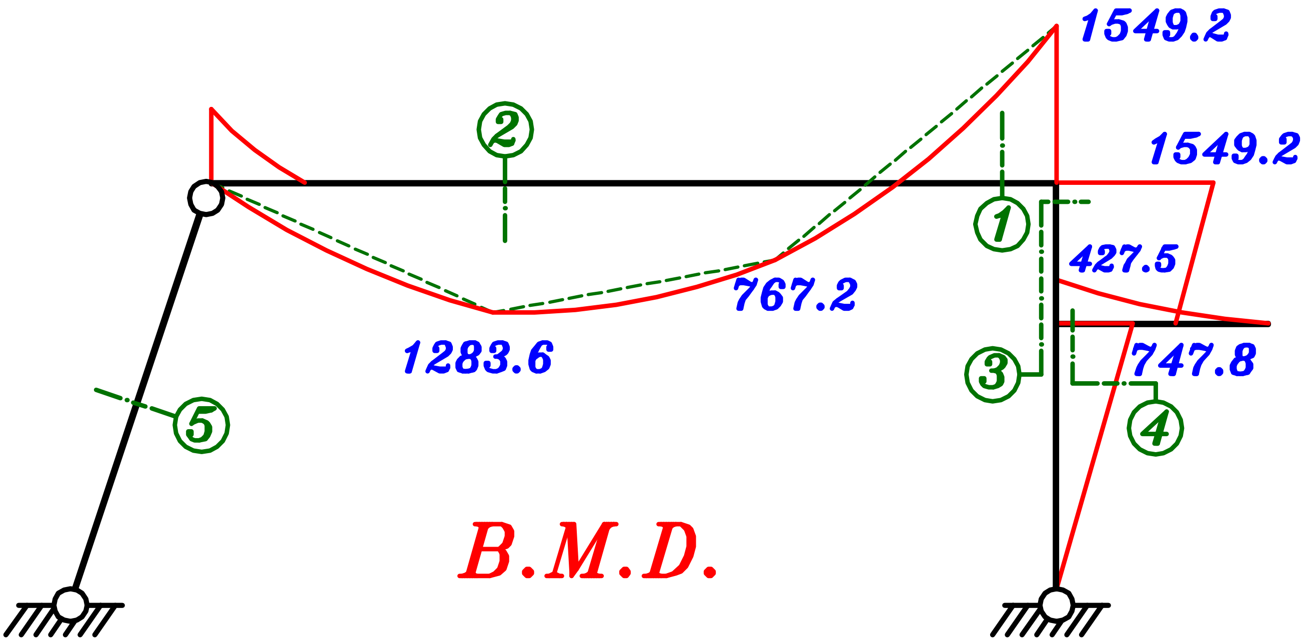


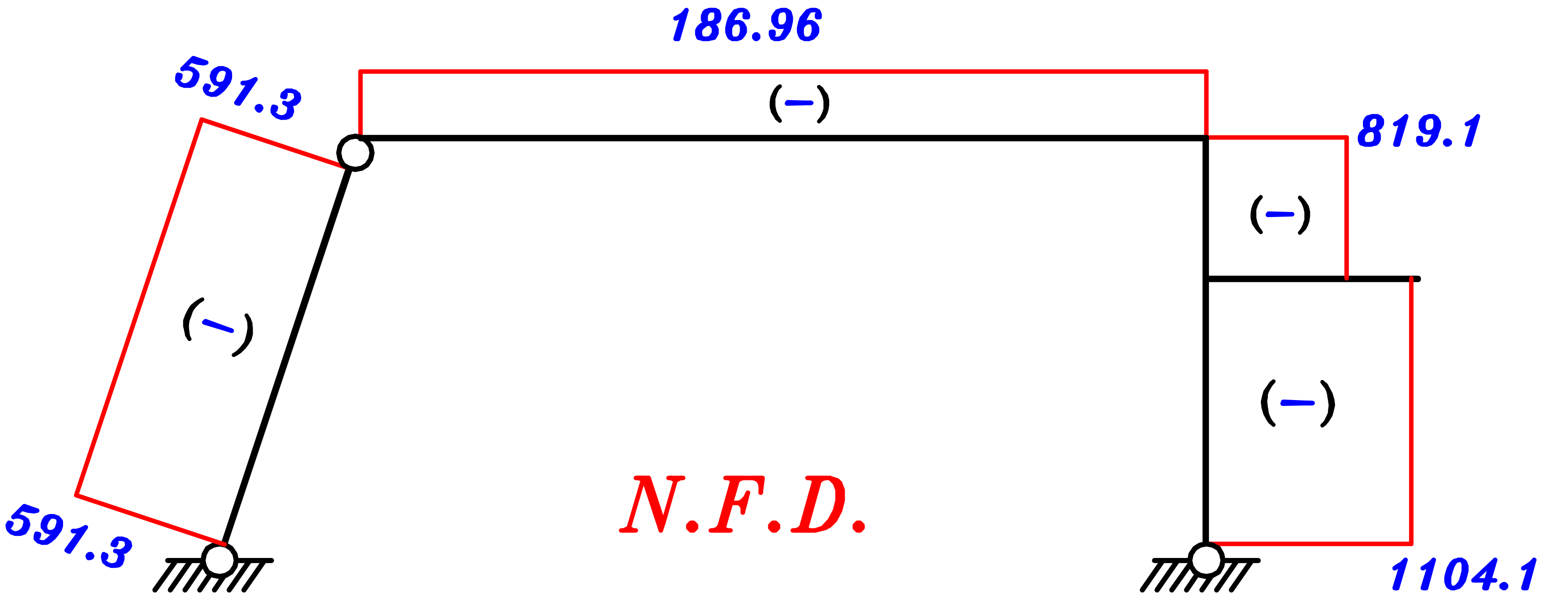
$\because \quad \sum M_a = Zero$

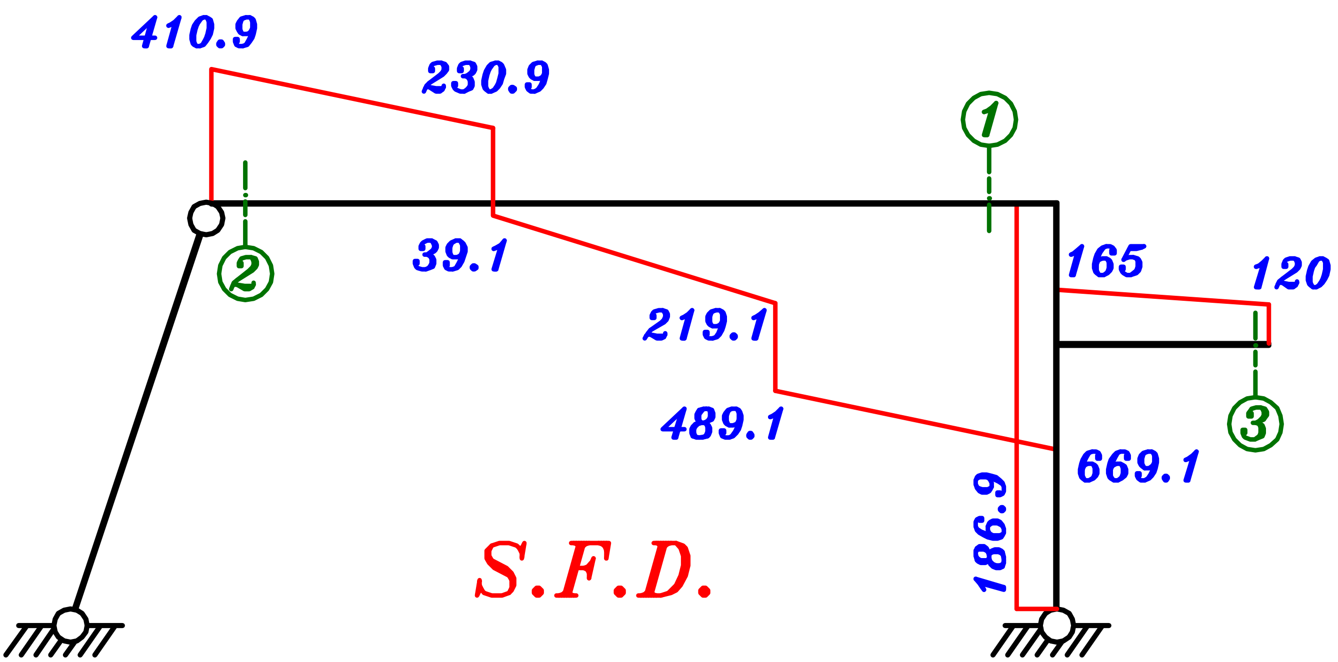
$\therefore \; 3X(12) + X(6) - 45(12)(6) - 150(12) - 270(8) - 270(4) + 120(3) + 15(3)(1.5) = Zero$

$X = 186.96 \text{ kN}$, $Y = 3X = 560.9 \text{ kN}$, $Y_1 = 1104.1 \text{ kN}$




1549.2
1549.2
427.5
767.2
1283.6
747.8
1
2
3
4
5
B.M.D.


186.96
591.3
(−)
819.1
(−)
(−)
(−)
591.3
1104.1
N.F.D.


410.9
230.9
1
39.1
2
165
120
219.1
489.1
3
669.1
186.9
S.F.D.

Design of Sections.

Sec. 1

$M = 1549.2$ kN.m , $P = 186.96$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{1549.2 * 10^6}{25 * 350}} = 1472.7 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1472.7) = (1619 \rightarrow 1913) \text{ mm}$$

Take $d = 1650$ mm , $t = 1650 + 100 = 1750$ mm

Check $$\frac{P}{F_{cu}\, b\, t} = \frac{186.96 * 10^3}{25 * 350 * 1750} = 0.012 < 0.04 \quad \therefore (\text{neglect } P)$$

$\because$ The sec. still R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

$\therefore$ Take $d = d_o = 1472.7$ mm

$\therefore$ Take $\boxed{d = 1500 \text{ mm}}$, $\boxed{t = 1600 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1549.2 * 10^6}{0.780 * 360 * 1472.7} = 3746.2 \text{ mm}^2$$

- Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 3746.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1500 = 1640.6 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3746.2 \text{ mm}^2$ $\quad 10\phi22$

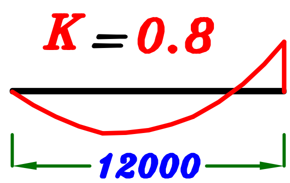
$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{22 + 25} = 6.91 = 6.0 \text{ bars}$$

Sec. ② $M = 1283.6$ kN.m , $P = 186.96$ kN , $b = 350$ mm

$d = 1500$ mm (the same depth of sec. ①)

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{186.96 * 10^3}{25 * 350 * 1500} = 0.014 < 0.04$ $\therefore$ (neglect P)

$\therefore$ The sec. will be T-sec. $\therefore$ use B



$$B = \begin{cases} C.L. - C.L. = Spacing = 6.0\ m = 6000\ mm \\ 16\ t_s + b = 16 * 120 + 350 = 2270\ mm \\ K \dfrac{L}{5} + b = 0.8 * \dfrac{12000}{5} + 350 = 2270\ mm \end{cases}$$

$\boxed{B = 2270\ mm}$

$\therefore d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, B}}$ $\therefore 1500 = C_1 \sqrt{\dfrac{1283.6 * 10^6}{25 * 2270}} \longrightarrow C_1 = 9.97 \longrightarrow J = 0.826$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1283.6 * 10^6}{0.826 * 360 * 1500} = 2877.7\ mm^2$$

– Check $A_{s_{min.}}$ $A_{s_{req.}} = 2877.7\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1500 = 1640.6\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2877.7\ mm^2$ 8 ϕ 22

Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (0.1 \rightarrow 0.2)\ 2877.7$ 4 ϕ 12

Sec. ③ R-Sec. $M = 1549.2$ kN.m , $P = 819.1$ kN

$$d_o = 3.5\sqrt{\frac{1549.2 * 10^6}{25 * 350}} = 1472.7 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1472.7) = (1619 \rightarrow 1913) \text{ mm}$$

Take $d = 1650$ mm , $\boxed{t = 1650 + 100 = 1750 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{819.1 * 10^3}{25 * 350 * 1750} = 0.053 > 0.04$ (Don't neglect P)

∴ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{1549.2}{819.1} = 1.89 \text{ m} \quad \therefore \frac{e}{t} = \frac{1.89}{1.75} = 1.08 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.89 + \frac{1.75}{2} - 0.10 = 2.665 \text{ m}$$

$$M_s = P * e_s = 819.1 * 2.665 = 2182.9 \text{ kN.m}$$

$$\therefore 1650 = C_1\sqrt{\frac{2182.9 * 10^6}{25 * 350}} \longrightarrow C_1 = 3.30 \longrightarrow J = 0.767$$

$$A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{2182.9 * 10^6}{0.767 * 360 * 1650} - \frac{819.1 * 10^3}{(360 \backslash 1.15)} = 2174.7 \text{ mm}^2$$

– Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2174.7$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1650 = 1084.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ ∴ Take $A_s = A_{s_{req.}} = 2174.7$ mm^2 6 Ø 22

Stirrup Hangers $\simeq 0.4\, A_s \simeq 0.4\,(2174.7) = 870$ mm^2 3 Ø 22

Sec. ④

$M = 427.5\ kN.m$, $b = 350\ mm$

$\because$ The sec. is R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

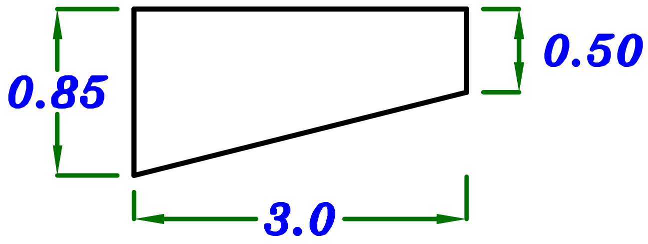
$$d = 3.5\sqrt{\frac{427.5 * 10^6}{25 * 350}} = 773.6\ mm \quad (\text{as R-Sec.})$$

$\therefore$ Take $\boxed{d = 800\ mm}$, $\boxed{t = 850\ mm}$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{427.5 * 10^6}{0.780 * 360 * 773.6} = 1968\ mm^2$$

6 ϕ22

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{850}{2} = 425\ mm \\ t_b = \frac{spacing}{12} = \frac{6000}{12} = 500\ mm \\ t - \frac{L_c}{3} = 850 - \frac{3000}{3} = -150\ mm \end{array} \right\} \quad Y = 500\ mm$$



Sec. ⑤

$(350*800)$ Axially Loaded Column. $P = 591.3\ kN$

$$\therefore P_{U.L.} = 0.35\ A_c\ F_{cu} + 0.67\ A_s\ F_y$$

$$\therefore 591.3 * 10^3 = 0.35\ (350 * 800)\ (25) + 0.67\ A_s\ (360)$$

$$\therefore A_s = -\ 7706\ mm^2 = (-\ Ve)\ \text{Value}$$

$$\therefore A_s = A_{s_{min.}} = \frac{0.8}{100} * 350 * 800 = 2240\ mm^2$$

10 ϕ18

Check Shear.

- **Allowable shear stress.**

- $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{25}{1.5}} = 0.98\ N\backslash mm^2$

- $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{25}{1.5}} = 2.85\ N\backslash mm^2$

Sec. 1

$Q = 669.1\ kN$

$$q_U = \frac{Q}{b\,d} = \frac{669.1 * 10^3}{350 * 1500} = 1.27\ N\backslash mm^2$$

$\therefore q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8\backslash m'$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n\,A_s(F_y\backslash\gamma_s)}{b\ \ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.27 - \frac{0.98}{2} = \frac{2 * 50.3\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 76.9\ mm < 100\ mm$$

* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$$1.27 - \frac{0.98}{2} = \frac{2 * 78.5\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 120\ mm > 100\ mm$$

$\therefore$ o.k.

$\therefore$ No. of stirrups$\backslash m' = \frac{1000}{S} = \frac{1000}{120} = 8.33 = 9.0$

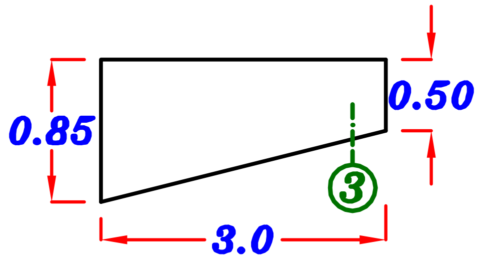
$\therefore$ Use Stirrups $\boxed{9\phi 10\backslash m'}$ 2 branches

Sec. ② $Q = 410.9$ kN

$$q_U = \frac{Q}{b\,d} = \frac{410.9 * 10^3}{350 * 1500} = 0.78 \ N\backslash mm^2$$

$\therefore\ q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m'}$

Sec. ③

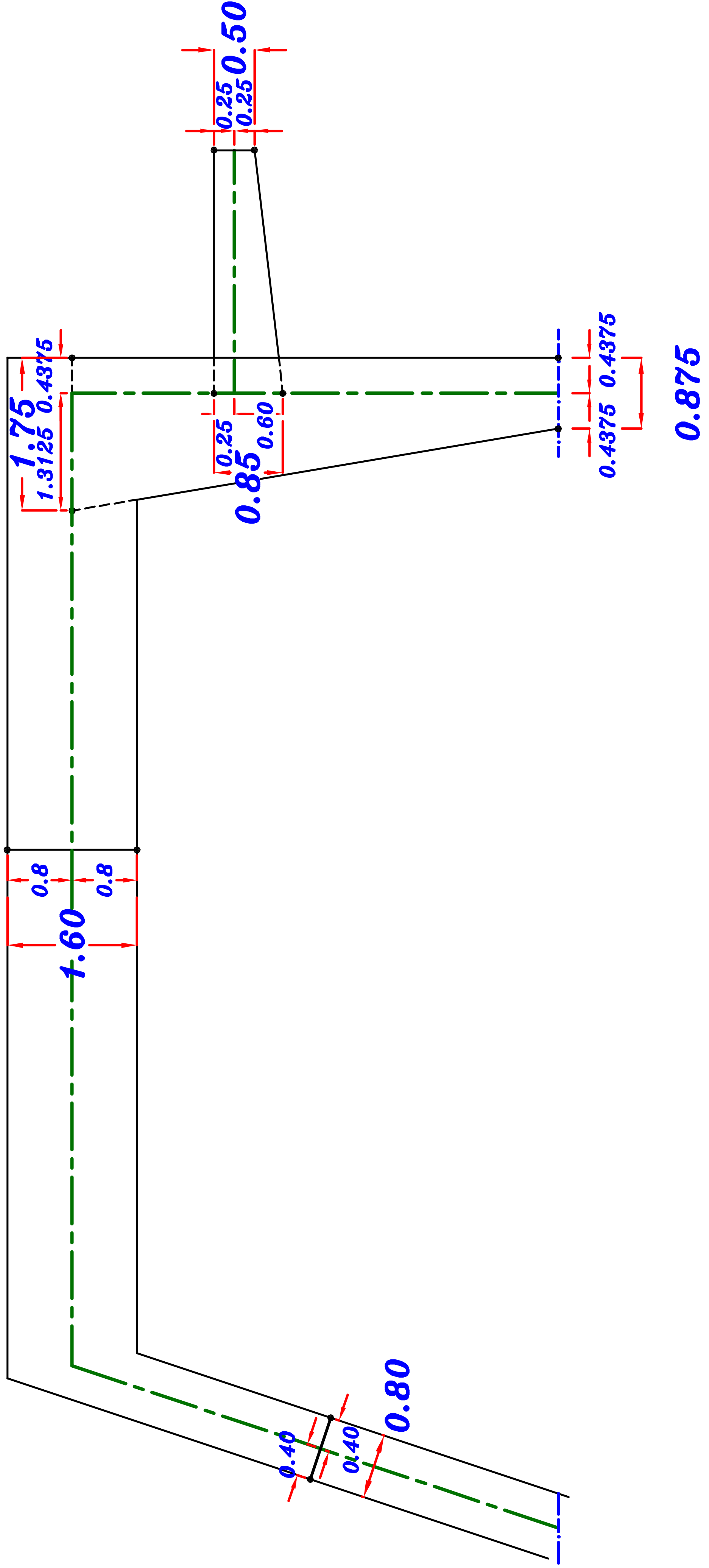


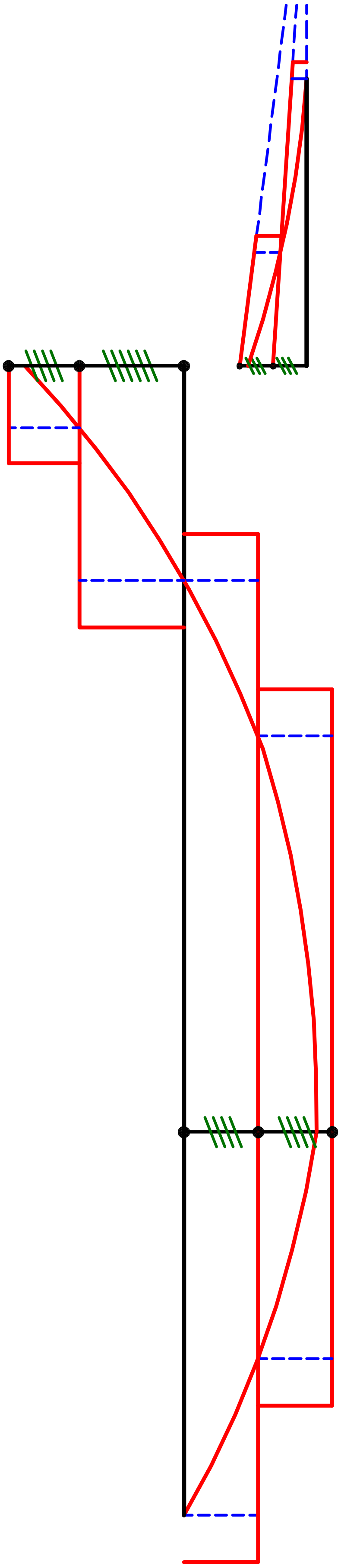
$Q = 120$ kN $\quad \tan\beta = 0.116$

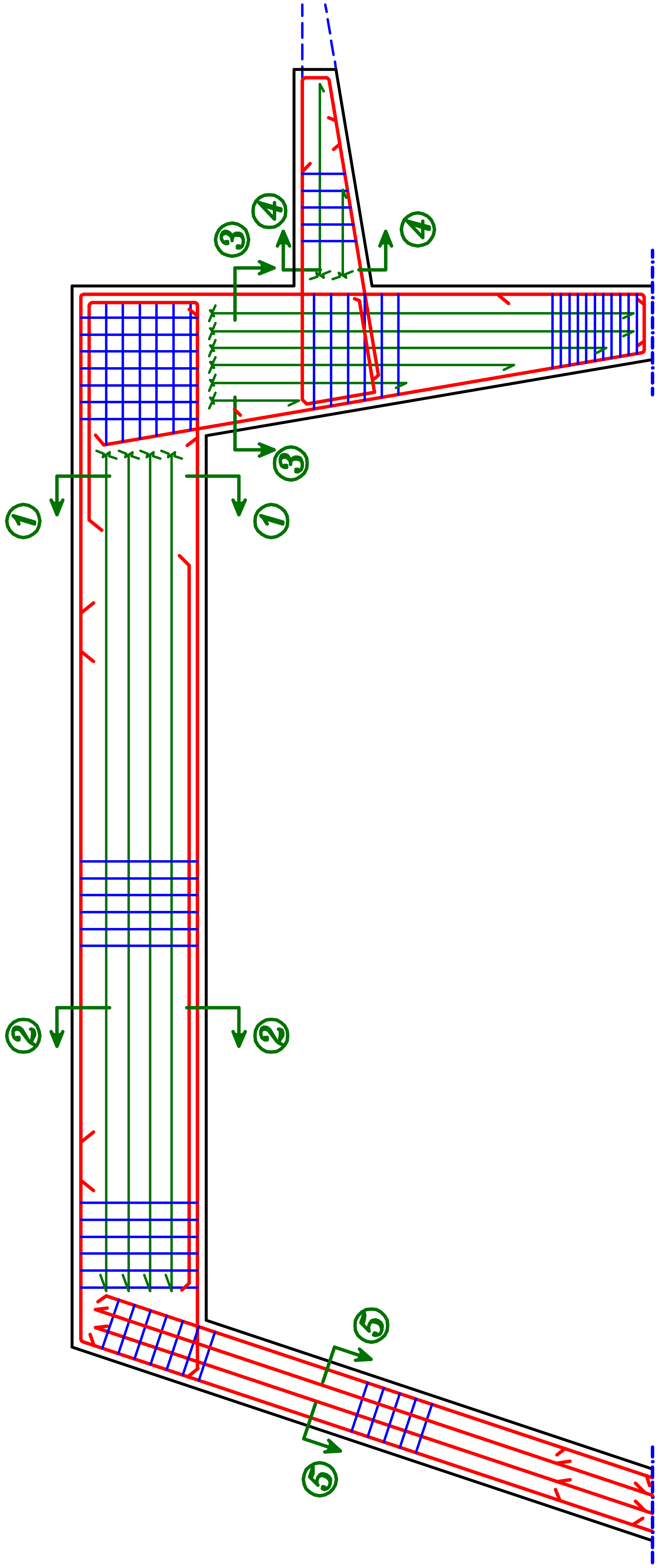
$$\therefore\ q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\ d^2}$$

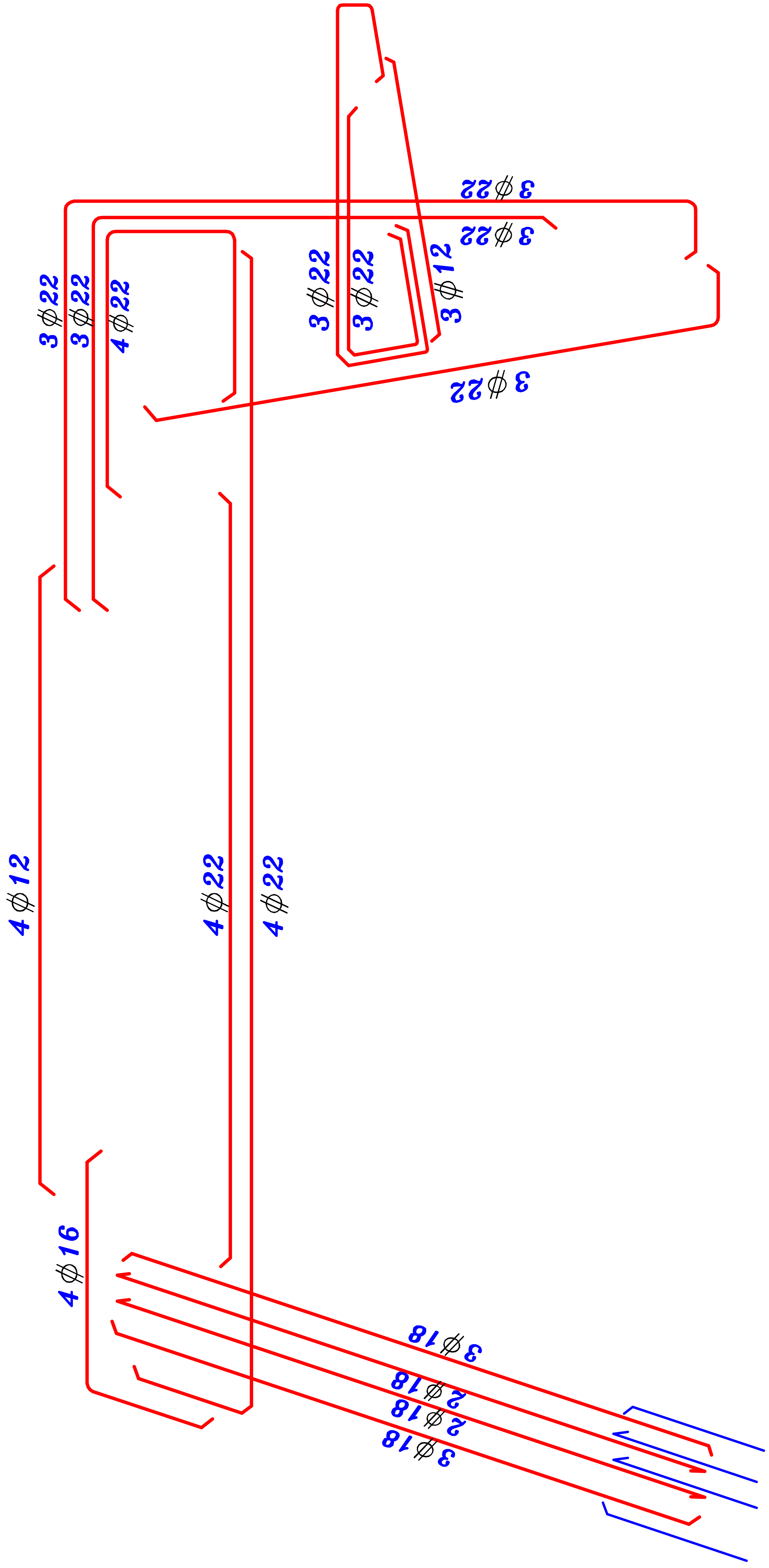
$$= \frac{120 * 10^3}{350 * 450} - ZERO = 0.761 \ N\backslash mm^2$$

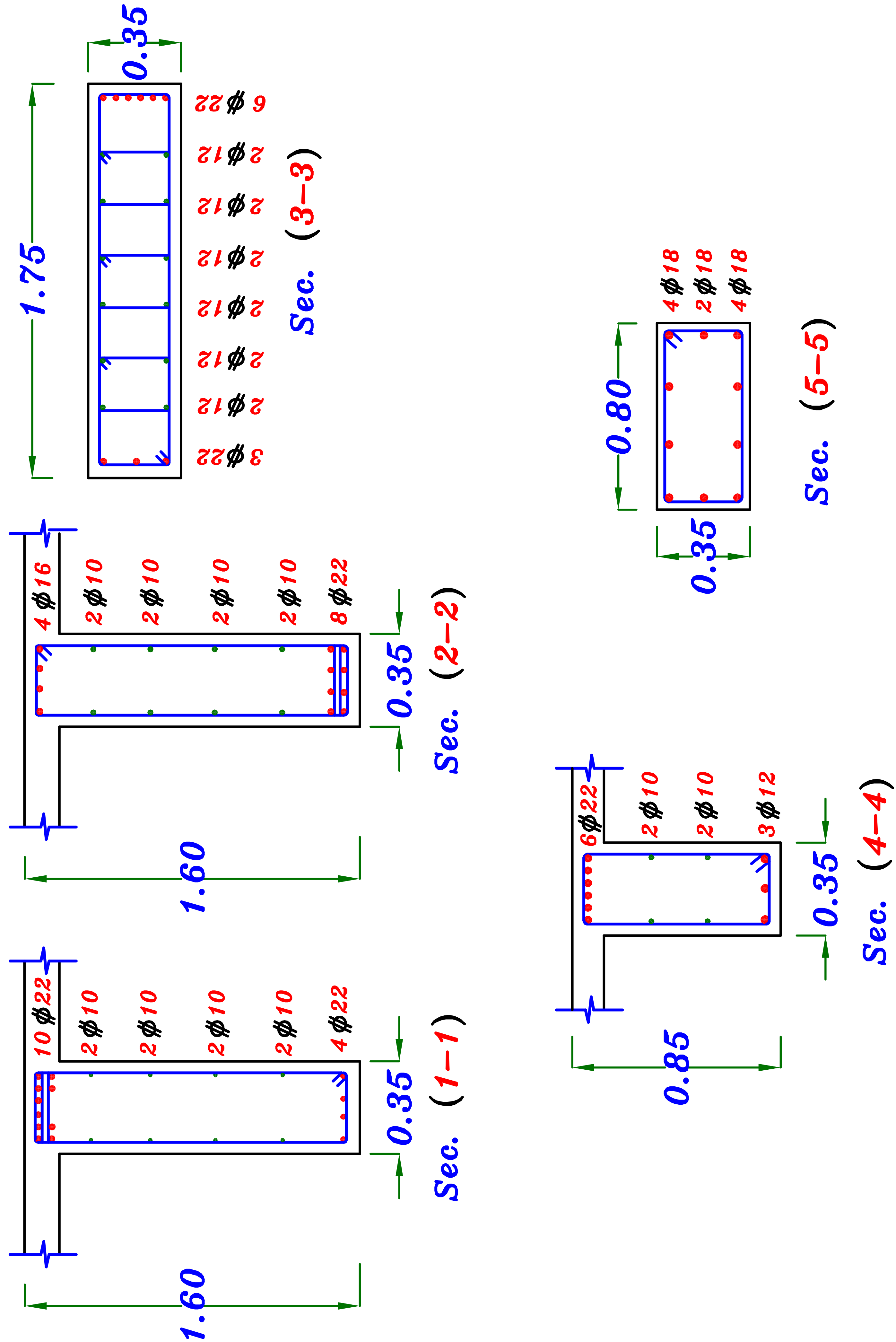
$\therefore\ q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m'}$


0.50
0.25
0.25
0.4375
1.75
1.3125
0.25
0.60
0.85
0.4375
0.4375
0.875
0.8
0.8
1.60
0.80
0.40
0.40

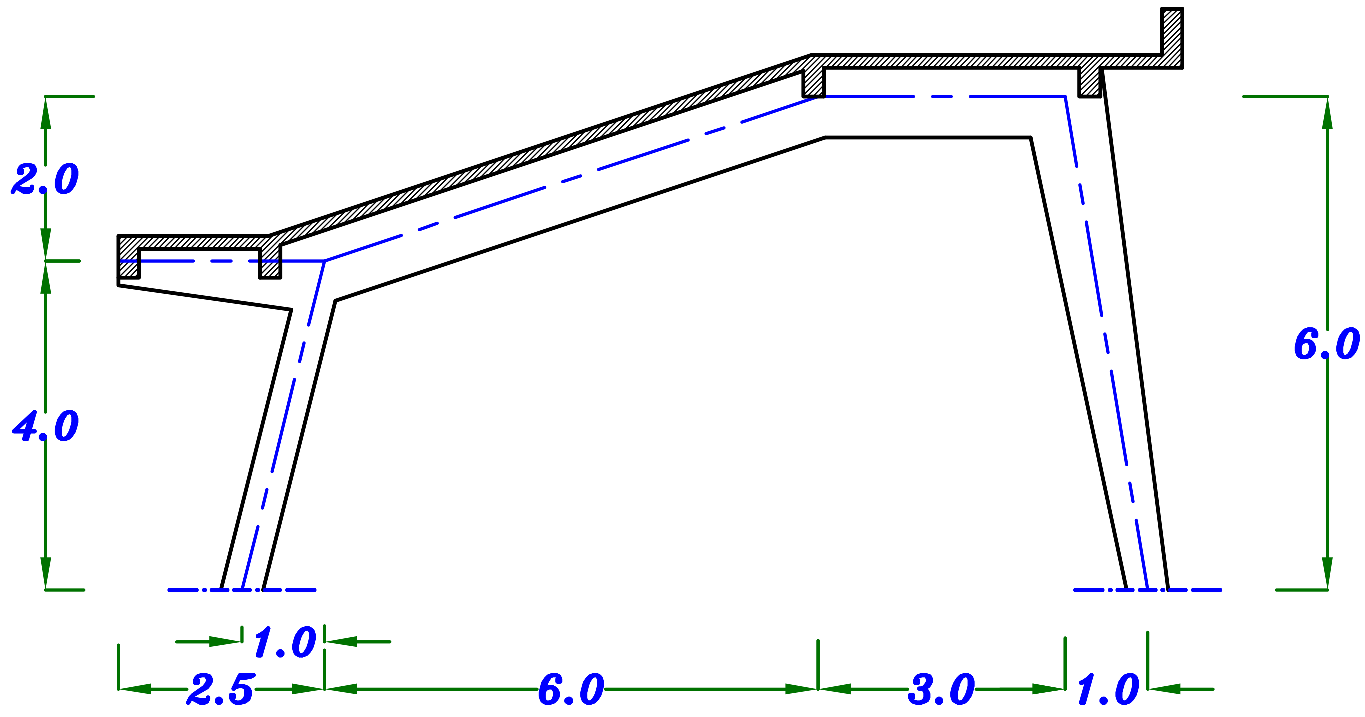



1
1
2
2
3
3
4
4
5
5


3 ϕ 22
3 ϕ 22
3 ϕ 22
3 ϕ 22
4 ϕ 22
3 ϕ 22
3 ϕ 22
3 ϕ 12
3 ϕ 22
4 ϕ 12
4 ϕ 22
4 ϕ 22
4 ϕ 16
3 ϕ 18
2 ϕ 18
2 ϕ 18
3 ϕ 18


0.35
1.75
6 Ø22
2 Ø12
2 Ø12
2 Ø12
2 Ø12
2 Ø12
2 Ø12
3 Ø22
Sec. (3-3)
4 Ø18
2 Ø18
4 Ø18
0.80
0.35
Sec. (5-5)
4 Ø16
2 Ø10
2 Ø10
2 Ø10
2 Ø10
8 Ø22
0.35
Sec. (2-2)
1.60
6Ø22
2 Ø10
2 Ø10
3 Ø12
0.35
Sec. (4-4)
0.85
10 Ø22
2 Ø10
2 Ø10
2 Ø10
2 Ø10
4 Ø22
0.35
Sec. (1-1)
1.60

Example.



Data.

$F_{cu} = 25 \; N\backslash mm^2$

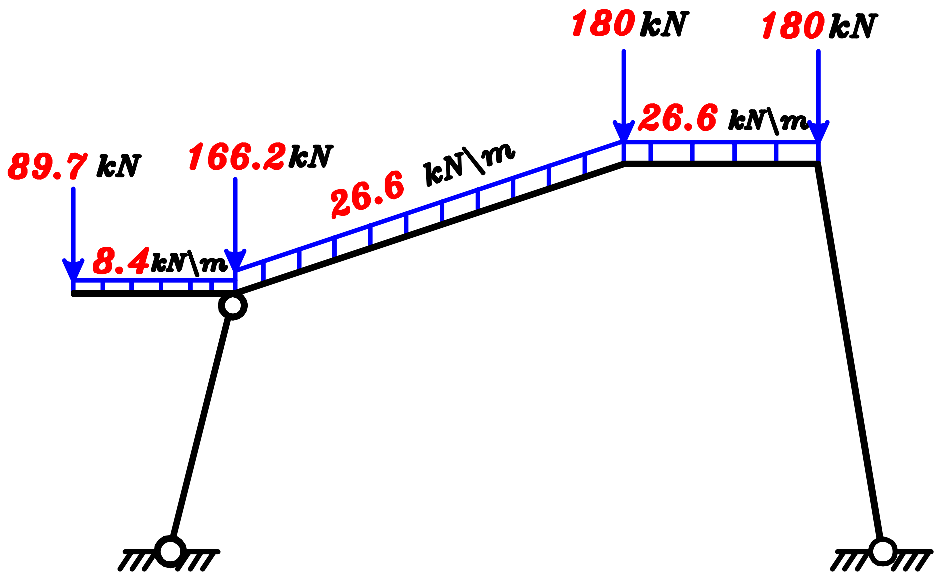
$F_y = 360 \; N\backslash mm^2$

$b_{(Beams)} = 250$ mm

$b_{(Frame)} = 350$ mm

$t_s = 140$ mm

Spacing = 6.0 m



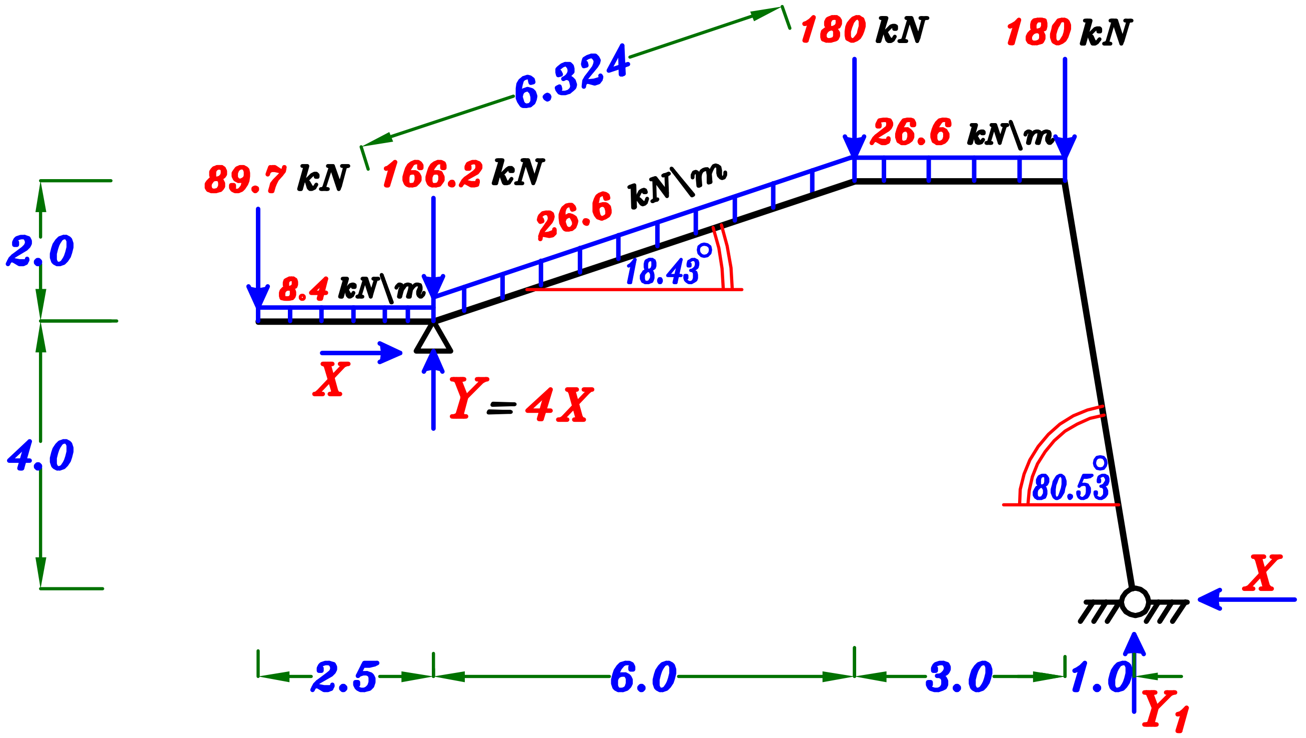
Req.

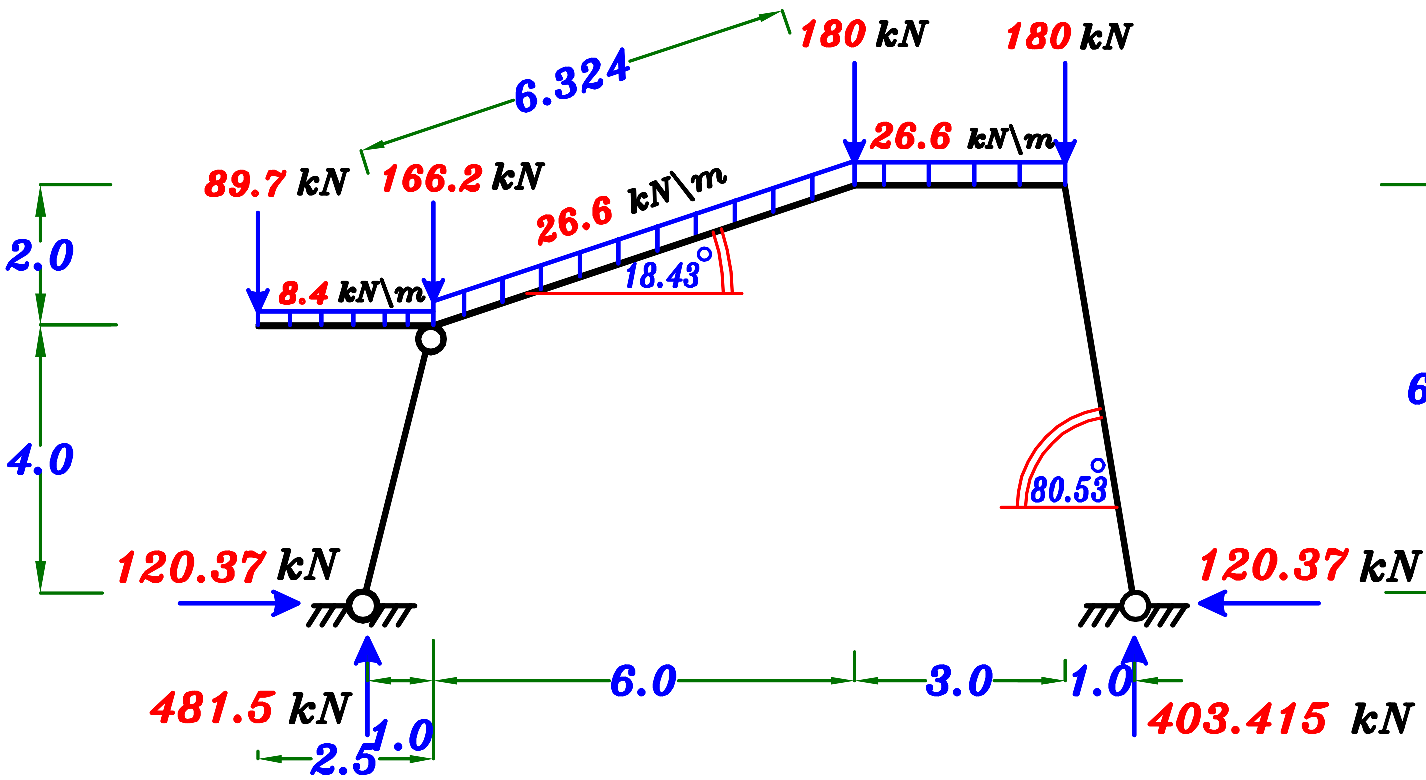
① Draw the Internal Forces Diagrams For the Frame due to the given working Loads. (Case of total Load is only required)

② Design the critical sections of the Frame. using U.L. design method in bending and shear.

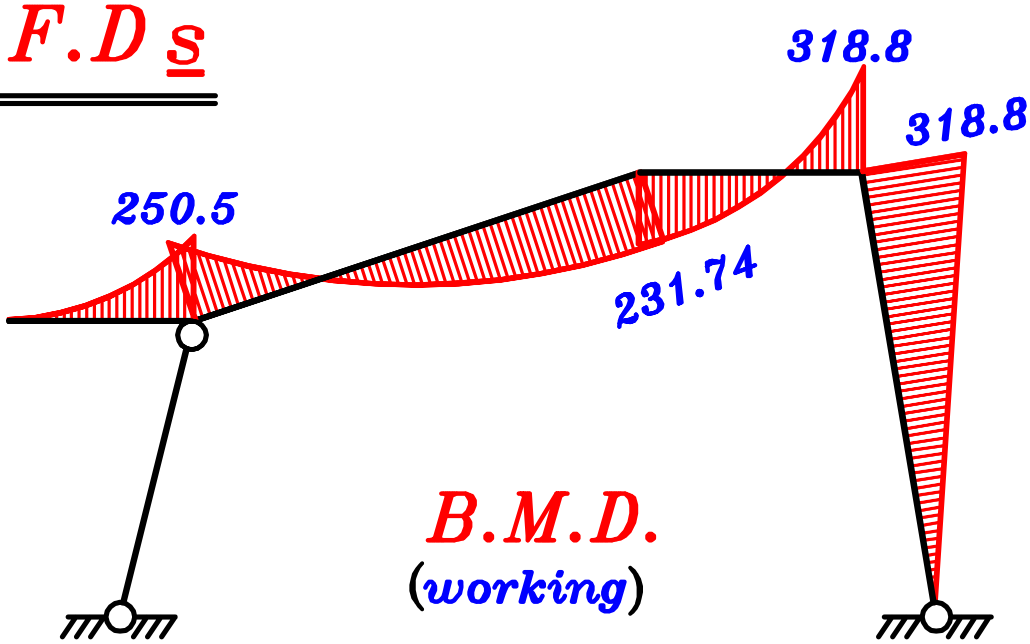
③ Draw Details of RFT. For Frame. in elevation to scale 1:50 and cross-section to scale 1:10 making curtailment of steel using Moment of Resistance Method.

Loads on the Frame.

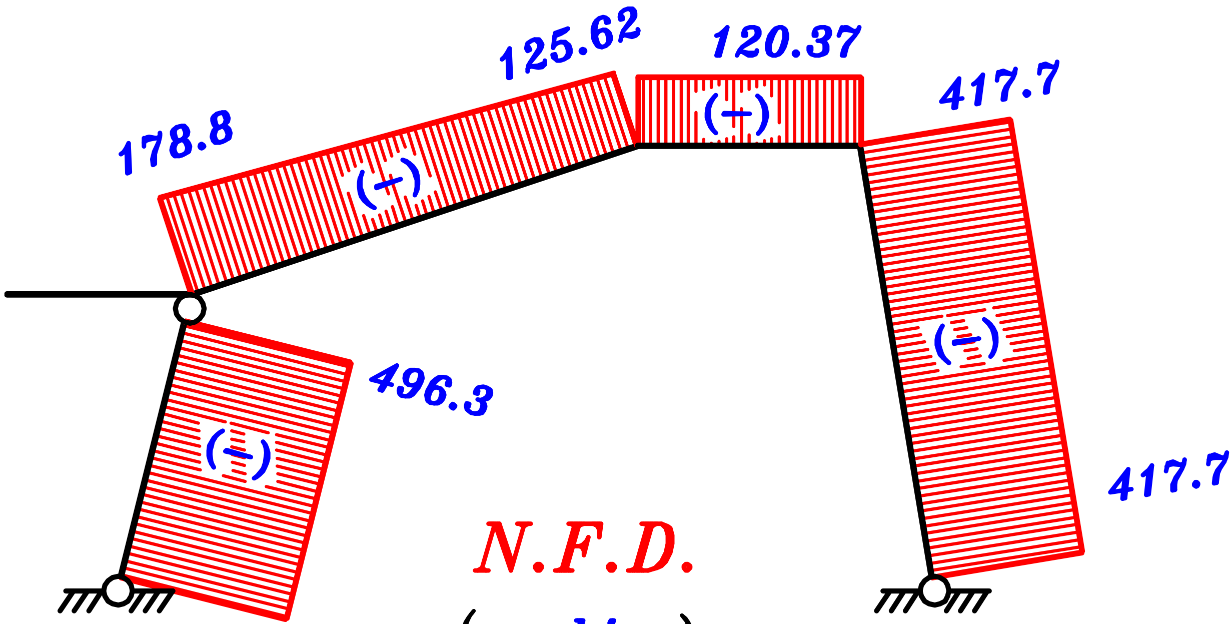




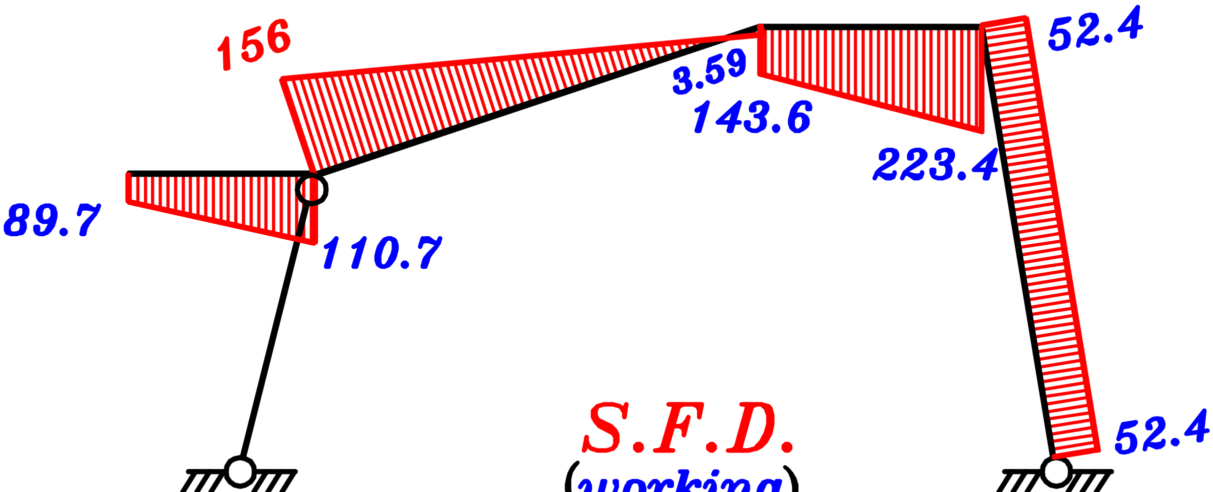
I.F.D s



B.M.D.
(working)

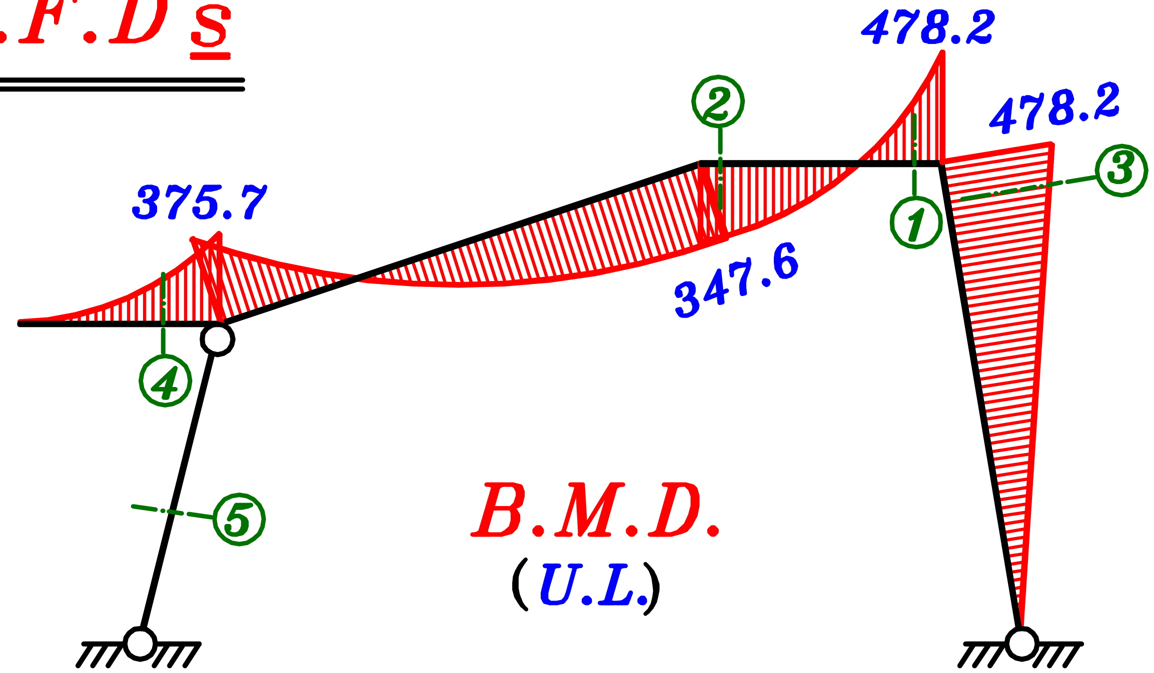


N.F.D.
(working)

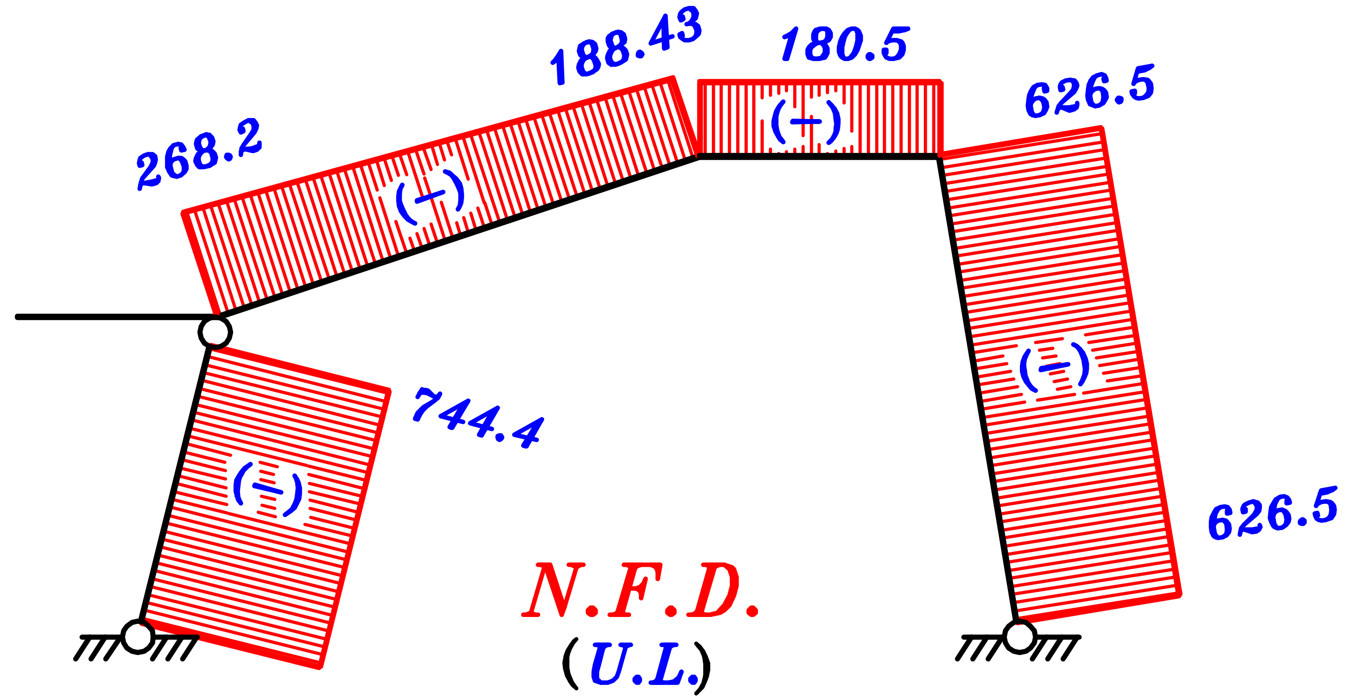


S.F.D.
(working)

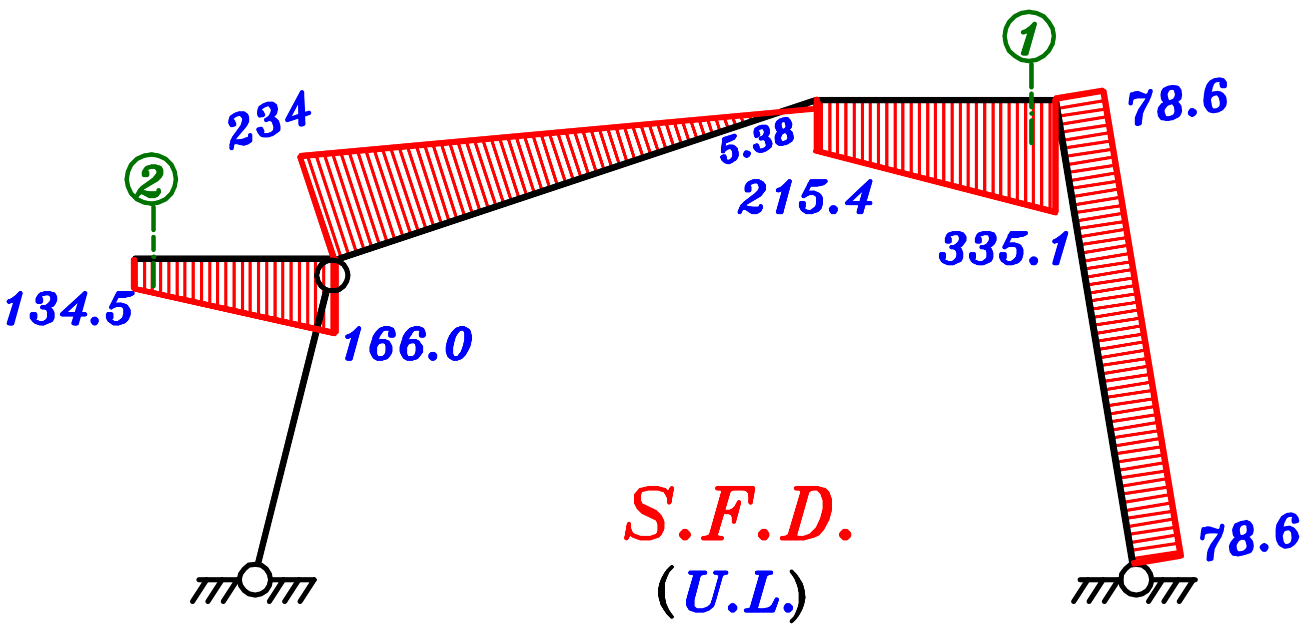
I.F.Ds



478.2
478.2
375.7
347.6
2
1
3
4
5
B.M.D.
(U.L.)



188.43
180.5
268.2
626.5
(-)
(+)
(-)
744.4
(-)
626.5
N.F.D.
(U.L.)



1
78.6
234
5.38
2
215.4
335.1
134.5
166.0
S.F.D.
(U.L.)
78.6

Design of Sections.

Sec. 1

$M = 478.2$ kN.m , $P = 180.5$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{478.2 * 10^6}{25 * 350}} = 818.2 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(818.2) = (900 \rightarrow 1063) \text{ mm}$$

Take $d = 950$ mm , $t = 950 + 50 = 1000$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{180.5 * 10^3}{25 * 350 * 1000} = 0.020 < 0.04$ $\therefore$ (neglect P)

$\therefore$ Take $d = d_o = 818.2$ mm

$\therefore$ Take $\boxed{d = 850 \text{ mm}}$, $\boxed{t = 900 \text{ mm}}$

$\because$ The sec. still R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{478.2 * 10^6}{0.780 * 360 * 818.2} = 2081.4 \text{ mm}^2$$

– Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2081.4$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 850 = 929.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2081.4$ mm^2 $\quad$ 11 ϕ 16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{16 + 25} = 7.92 = 7.0 \text{ bars}$$

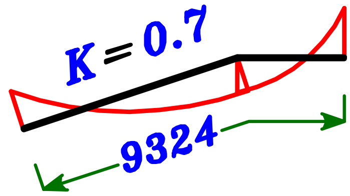
Sec. ②

$M = 347.6 \text{ kN.m}$, $P = 186.3 \text{ kN}$, $b = 350 \text{ mm}$

$d = 850 \text{ mm}$ (the same depth of Sec. ①)

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{186.3 * 10^3}{25 * 350 * 850} = 0.025 < 0.04$ $\therefore$ (neglect P)

$\therefore$ The sec. will be T-sec. $\therefore$ use B



$$B = \begin{cases} C.L. - C.L. = \text{Spacing} = 6.0 \text{ m} = 6000 \text{ mm} \\ 16\, t_s + b = 16 * 140 + 350 = 2590 \text{ mm} \\ K\dfrac{L}{5} + b = 0.7 * \dfrac{9324}{5} + 350 = 1655.3 \text{ mm} \end{cases}$$

$$\boxed{B = 1655.3 \text{ mm}}$$

$\therefore d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, B}}$ $\therefore 850 = C_1 \sqrt{\dfrac{347.6 * 10^6}{25 * 1655.3}} \longrightarrow C_1 = 9.27 \longrightarrow J = 0.826$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{347.6 * 10^6}{0.826 * 360 * 850} = 1375.2 \text{ mm}^2$$

- <u>Check $A_{s_{min.}}$</u> $A_{s_{req.}} = 1375.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 850 = 929.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1375.2 \text{ mm}^2$ 7 ∅ 16

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 1375.2$ 2 ∅ 12

Sec. ③ R-Sec. $M = 478.2$ kN.m , $P = 626.5$ kN

$$d_o = 3.5\sqrt{\frac{478.2 * 10^6}{25 * 350}} = 818.2 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(818.2) = (900 \rightarrow 1063) \text{ mm}$$

Take $d = 950$ mm , $\boxed{t = 950 + 50 = 1000 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{626.5 * 10^3}{25 * 350 * 1000} = 0.071 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{478.2}{626.5} = 0.763 \text{ m} \quad \therefore \frac{e}{t} = \frac{0.763}{1.0} = 0.763 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 0.763 + \frac{1.0}{2} - 0.05 = 1.213 \text{ m}$$

$$M_s = P * e_s = 626.5 * 1.213 = 759.9 \text{ kN.m}$$

$$\therefore 950 = C_1\sqrt{\frac{759.9 * 10^6}{25 * 350}} \longrightarrow C_1 = 3.22 \longrightarrow J = 0.764$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \delta_s)} = \frac{759.9 * 10^6}{0.764 * 360 * 950} - \frac{626.5 * 10^3}{(360 \backslash 1.15)} = 907 \text{ mm}^2$$

- Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 907 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 950 = 1039 \text{ mm}^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$\left.\begin{aligned} A_{s_{min.}} &= \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 950 = 1039 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} &= 1.3 * 907 = 1179.1 \text{ mm}^2 \end{aligned}\right\} \text{الأقل} = 1039 \text{ mm}^2 \quad 6\phi16$$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{16 + 25} = 7.92 = 7.0$$

Stirrup Hangers $\simeq 0.4\, A_s \simeq 0.4\,(1039) = 416 \text{ mm}^2$ $\quad 2\phi18$

Sec. ④

$M = 375.7$ kN.m , $b = 350$ mm

$\because$ The sec. is R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

$$d = 3.5\sqrt{\frac{375.7 * 10^6}{25 * 350}} = 725.2 \text{ cm.} \quad (\text{as R-Sec.})$$

$\therefore$ Take $\boxed{d = 750 \text{ mm}}$, $\boxed{t = 800 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{375.7 * 10^6}{0.780 * 360 * 725.2} = 1844.9 \text{ mm}^2$$

- Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 1844.9 \text{ mm}^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 750 = 820.3 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1844.9 \text{ mm}^2$ 10ϕ16

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{800}{2} = 400 \text{ mm} \\ t_b = \frac{\text{spacing}}{12} = \frac{6000}{12} = 500 \text{ mm} \\ t - \frac{L_c}{3} = 800 - \frac{2500}{3} = -33.3 \text{ mm} \end{array} \right\} \quad Y = 500 \text{ mm}$$

Sec. ⑤

(350*450) Axially Loaded Column. $P = 744.4$ kN

$$\therefore P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$$

$$\therefore 744.4 * 10^3 = 0.35\,(350 * 450)(25) + 0.67\,A_s\,(360)$$

$\therefore A_s = -2627 \text{ mm}^2$ = (−Ve) Value

$$\therefore A_s = A_{s_{min.}} = \frac{0.8}{100} * 350 * 450 = 1260 \text{ mm}^2$$ 12ϕ12

Check Shear.

- Allowable shear stress.

- $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\delta_c}} = 0.24\sqrt{\frac{25}{1.5}} = 0.98 \quad N\backslash mm^2$

- $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\delta_c}} = 0.7\sqrt{\frac{25}{1.5}} = 2.85 \quad N\backslash mm^2$

Sec. ① $Q = 335.1$ kN

$\therefore$ Actual shear stress. $= q_U = \frac{Q}{b\,d} = \frac{335.1 * 10^3}{350 * 850} = 1.126 \quad N\backslash mm^2$

$\therefore q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8\backslash m'$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n\,A_s(F_y\backslash \delta_s)}{b\;S}$

* Take $n = 2$, $\phi\,8 \longrightarrow A_s = 50.3 \;\; mm^2$

$1.126 - \frac{0.98}{2} = \frac{2 * 50.3\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 94.32 \; mm < 100 \; mm$

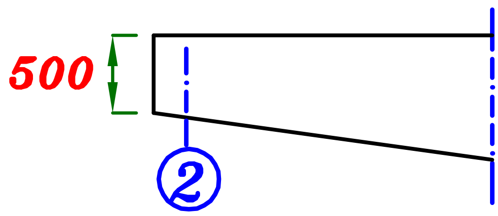
* Take $n = 2$, $\phi\,10 \longrightarrow A_s = 78.5 \; mm^2$

$1.126 - \frac{0.98}{2} = \frac{2 * 78.5\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 147.19 \; mm > 100 \; mm \;\; \therefore$ o.k.

$\therefore$ No. of stirrups$\backslash m' = \frac{1000}{S} = \frac{1000}{147.19} = 6.80 = 7.0$

$\therefore$ Use Stirrups $\boxed{7\phi 10 \backslash m'}$ 2 branches

Sec. ② $Q = 134.5$ kN

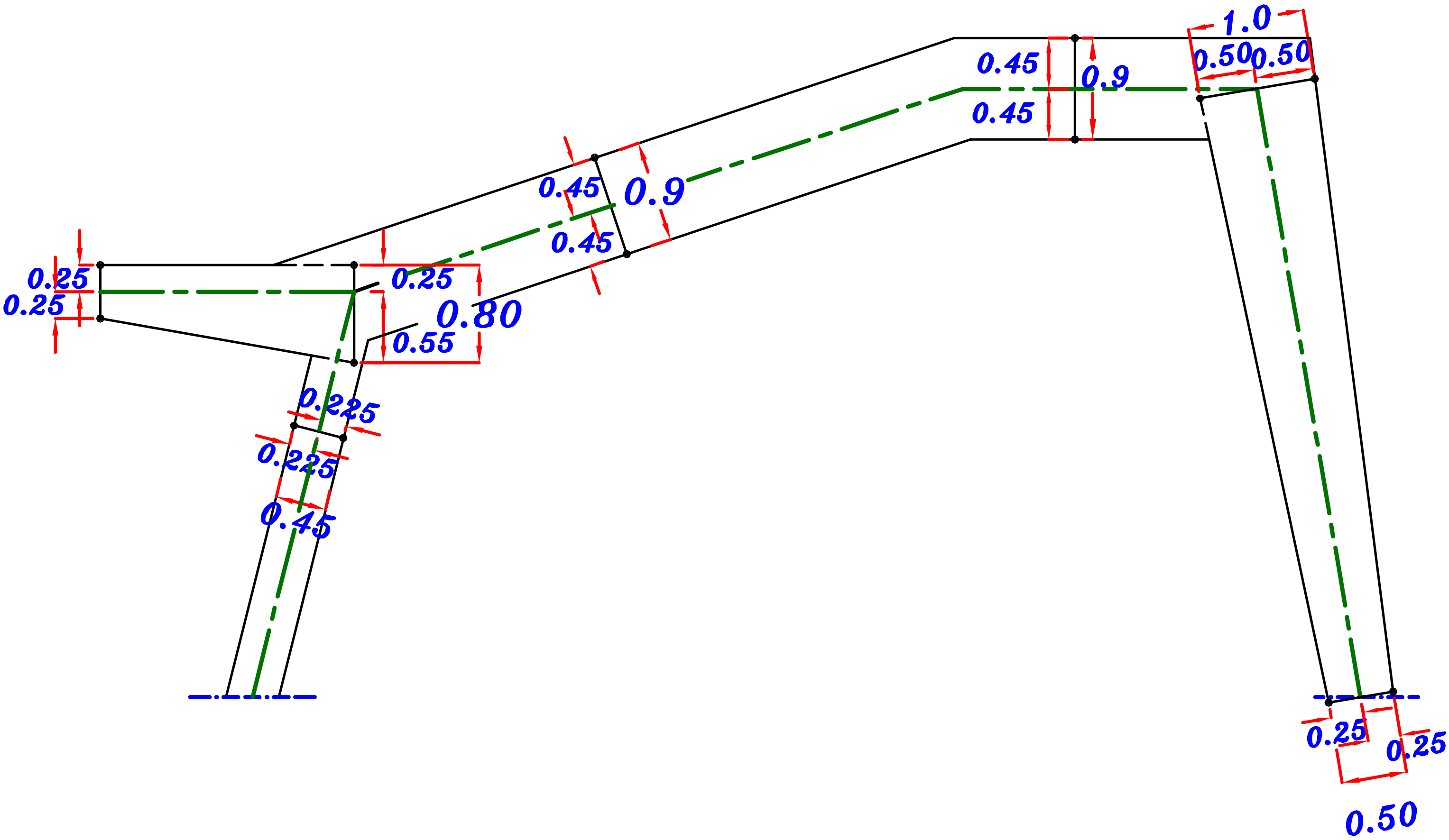


$\therefore$ Actual shear stress. $=$

$$q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\;d^2}$$

$= \frac{134.5 * 10^3}{350 * 450} - \text{ZERO} = 0.853 \quad N\backslash mm^2$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m'}$



مراحل رسم التسليح لل *Frame* مع مراعاه الترتيب

١- رسم مكان التسليح الرئيسى جهة ال *moment*

٢- تسليح ال *Joints* (عمل التباعد و التداخل)

٣- الحديد السفلى يرسم من وش العمود الى وش العمود

٤- رسم التسليح عند نهاية الاعمده (عمل أشاير أو لف الحديد فى العمود)

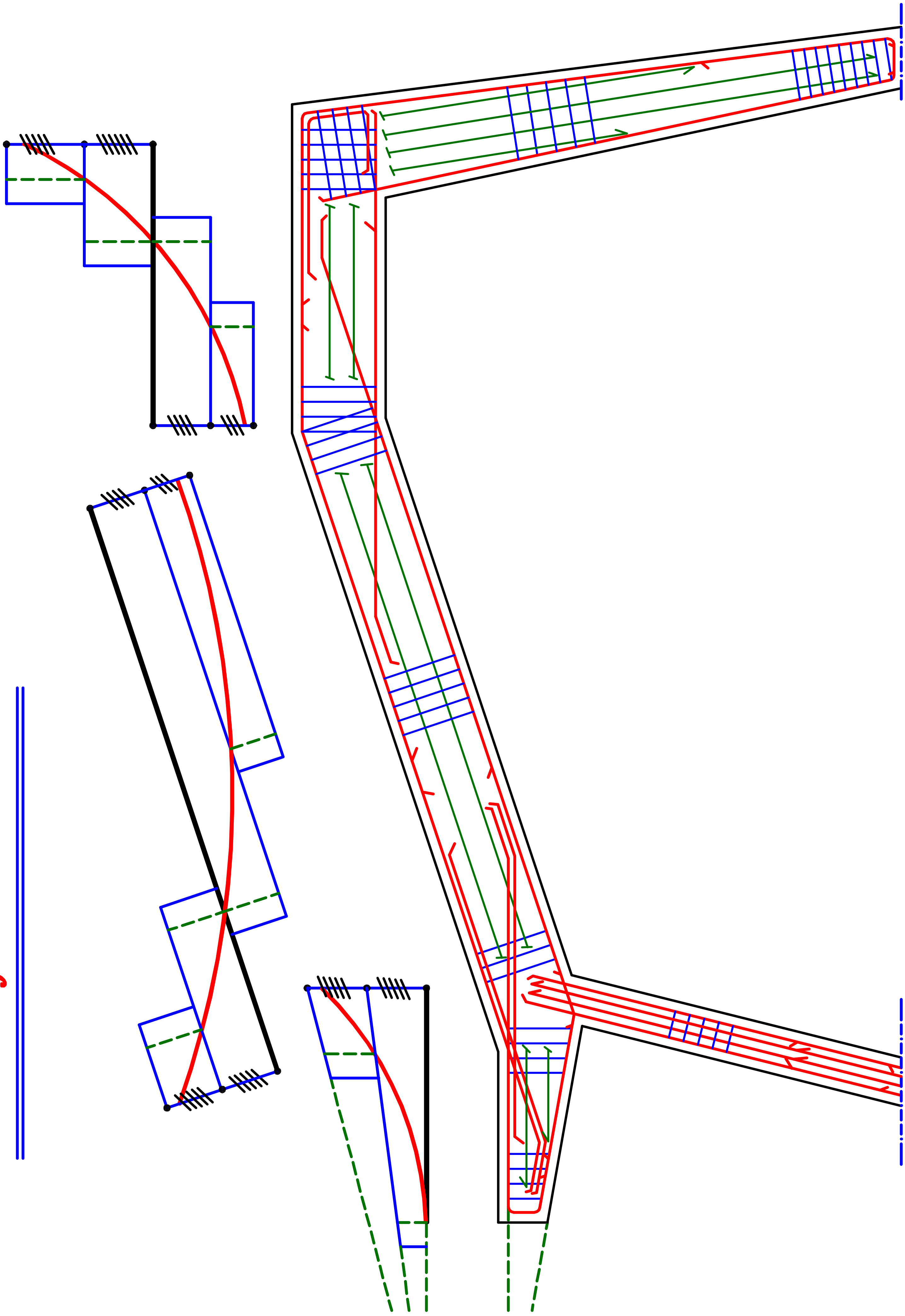
٥- رسم البلوكات مع مراعاه عدد الاسياخ و عدد الصفوف

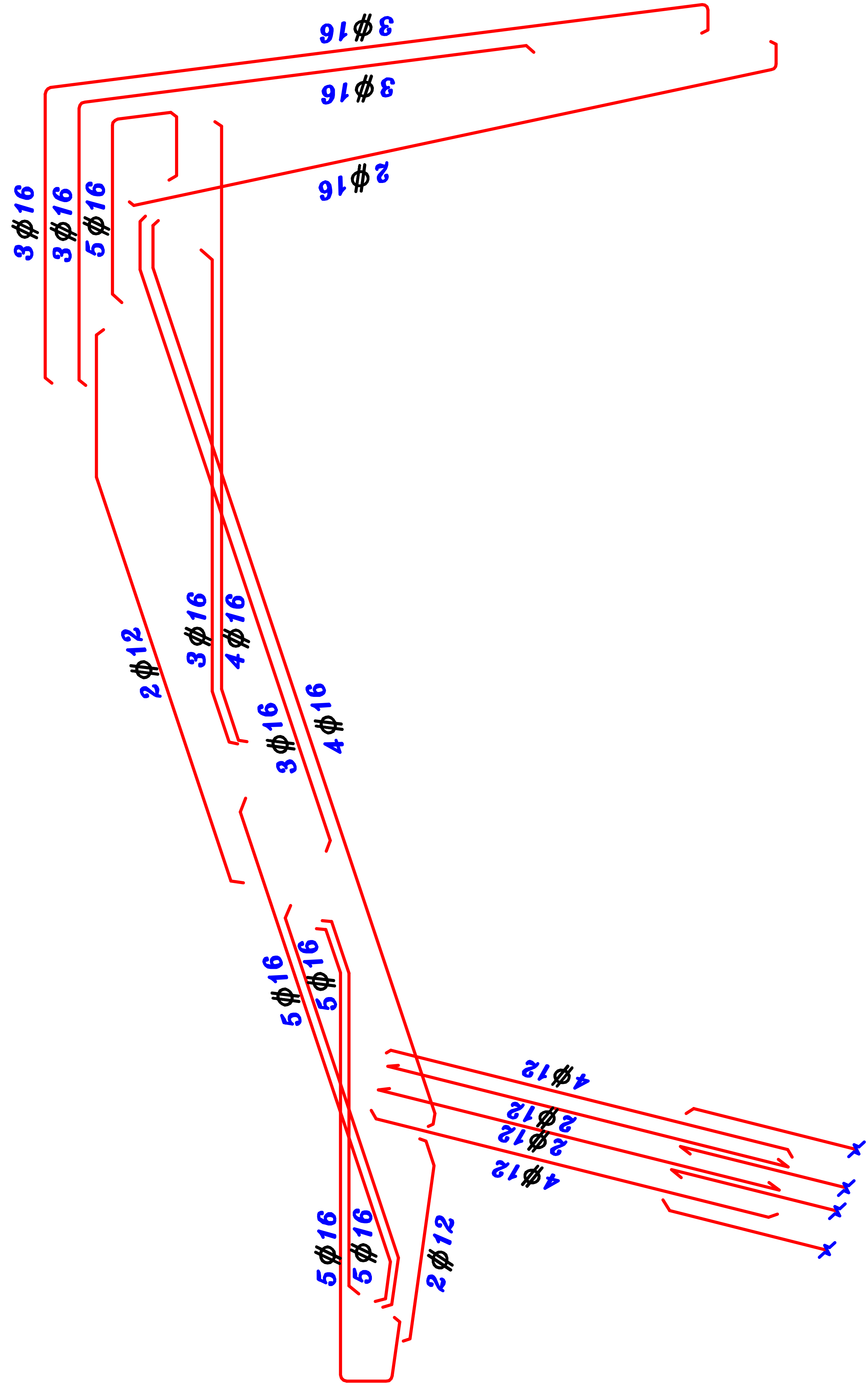
٦- و ضع ال *Stirrup Hangers*

٧- و ضع الكانات و ال *Shrinkage bars* و ال *Buckling bars*

٨- رسم التفريد و ال *Sections*

R.F.T. of the Frame.




3 φ16
3 φ16
2 φ16
3 φ16
3 φ16
5 φ16
2 φ12
3 φ16
4 φ16
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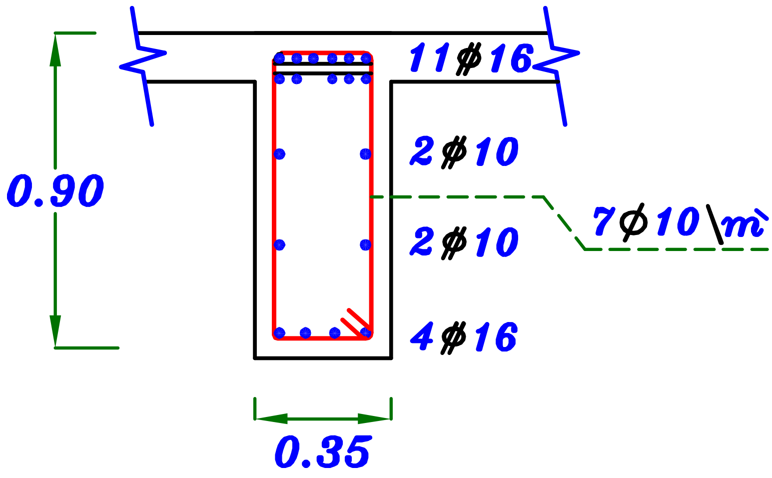
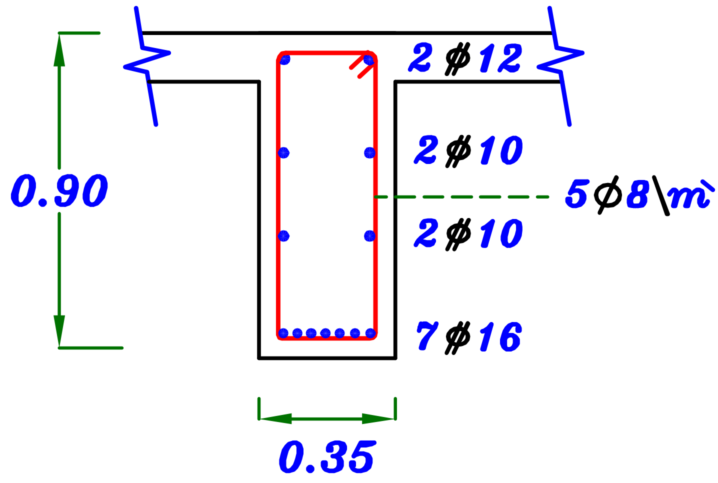
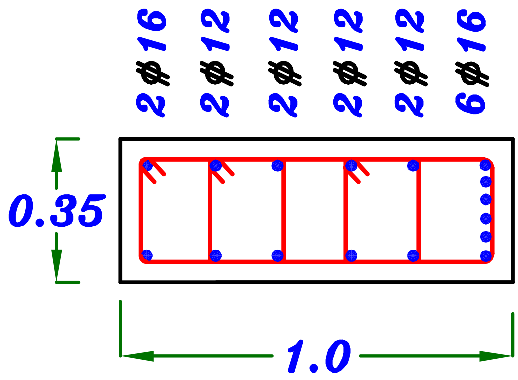
5 φ16
5 φ16
2 φ12



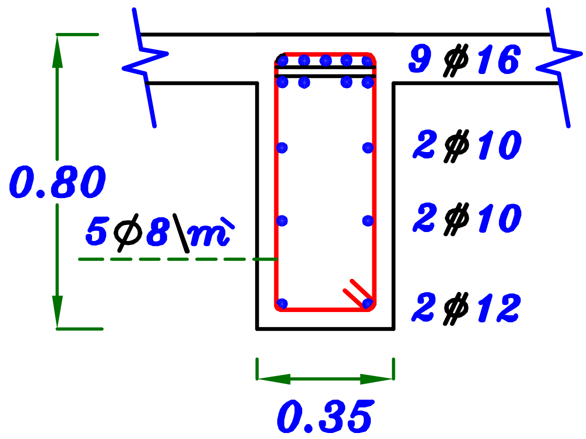
Sec. (1–1)



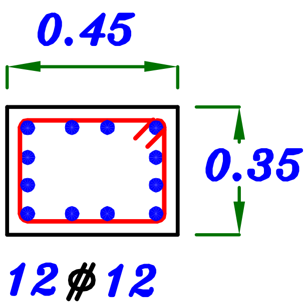
Sec. (2–2)



Sec. (3–3)



Sec. (4–4)



Sec. (5–5)

Example.

The Figure shows a sectional elevation of an intermediate two hinged Frame (a b c d d\`c\`b\`a\`) of a shed. The shed is composed of a group of beams and Frames.

For the given working loads and reactions it is required to :

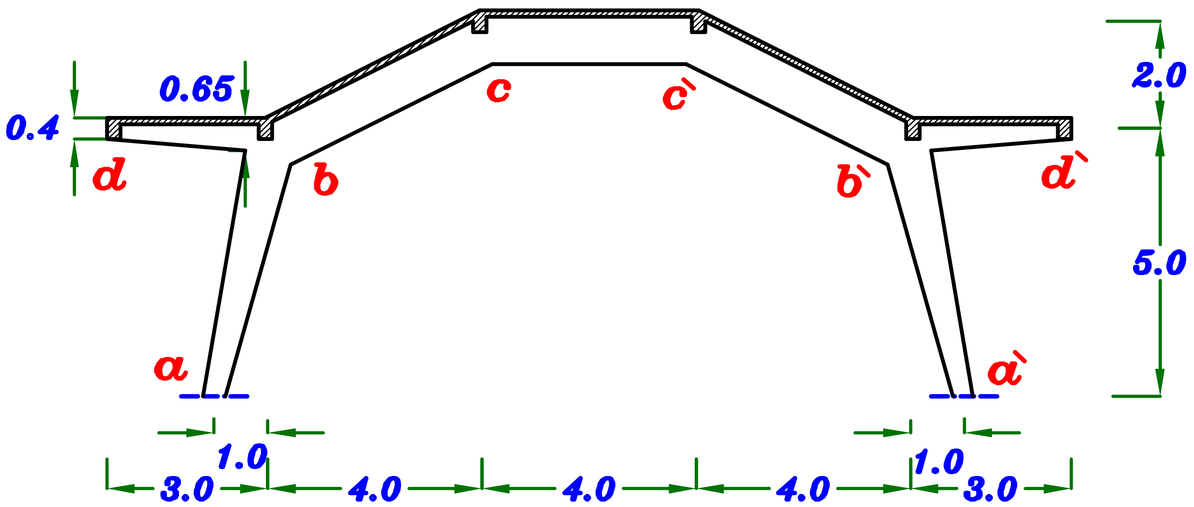
1) Draw the shearing, Normal Force and Bending Moment diagrams For the Frame.

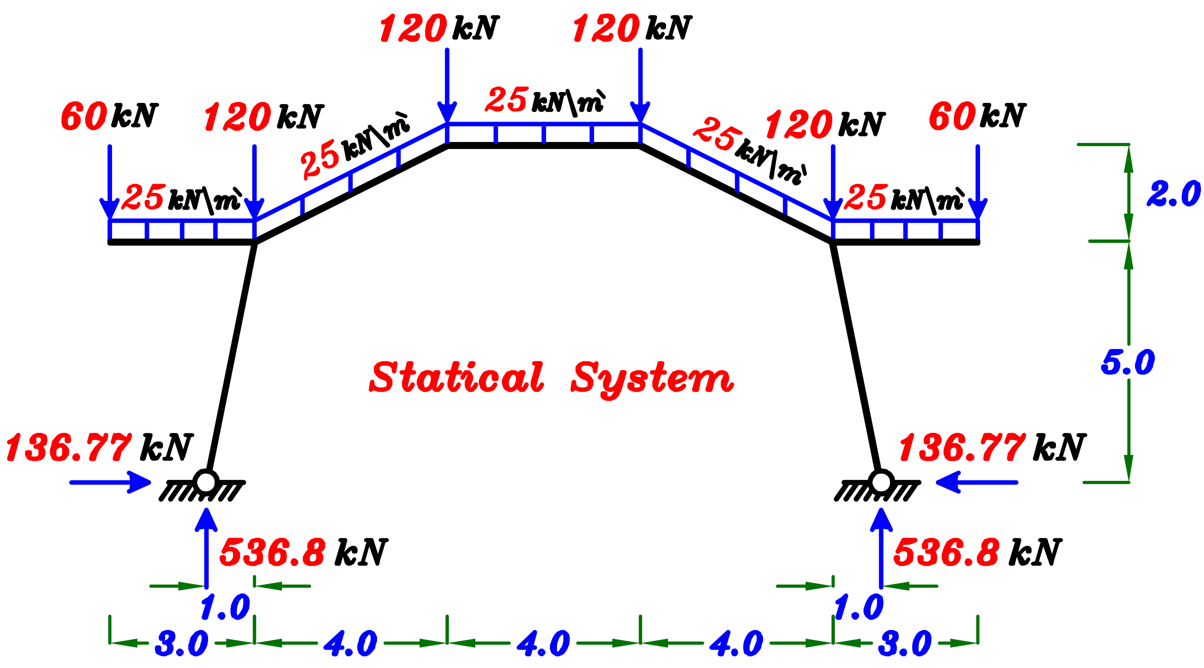
2) Using the ultimate limit design method, design the critical sections For an intermediate Frame to satisfy the requirements of the internal Forces. using the dimensions given For the part (b d).

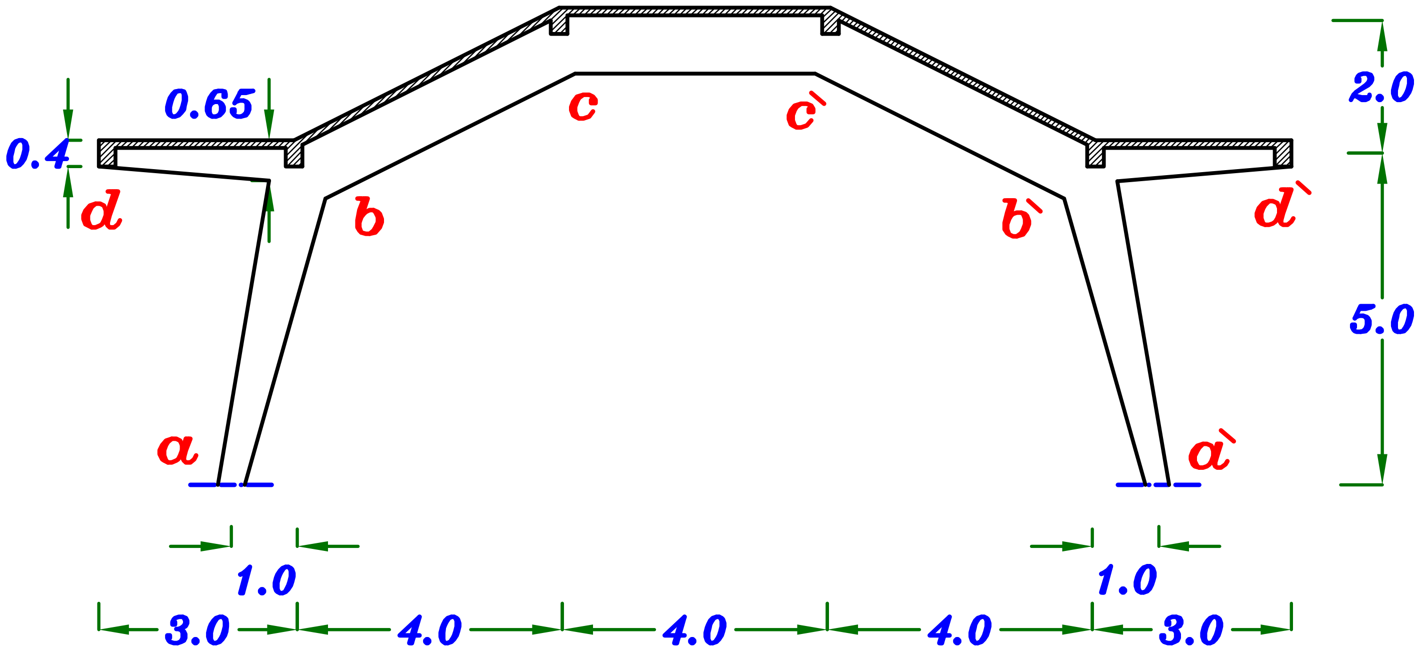
3) Complete detailed drawings (i.e. elevation and sections) showing all necessary reinforcement using moment of resistance diagram.

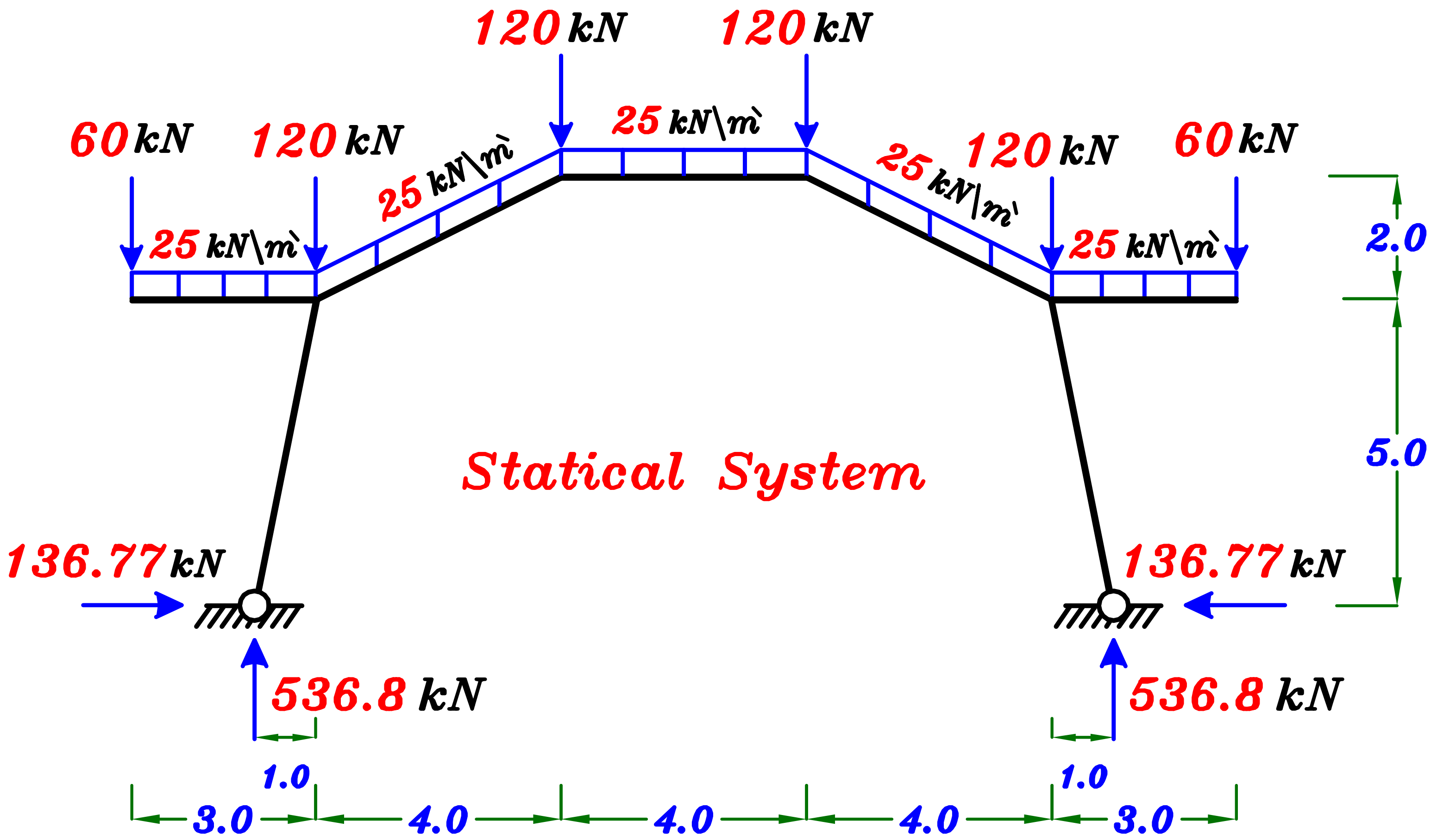
Data to be used:

- The spacing between Frames is 5.0 m center to center.
- Slab thickness t_s = 120 mm
- Breadth of all beams = 250 mm and Frames = 350 mm
- Concrete characteristic strength F_{cu} = 25 N\mm^2 and steel of grade 360/520

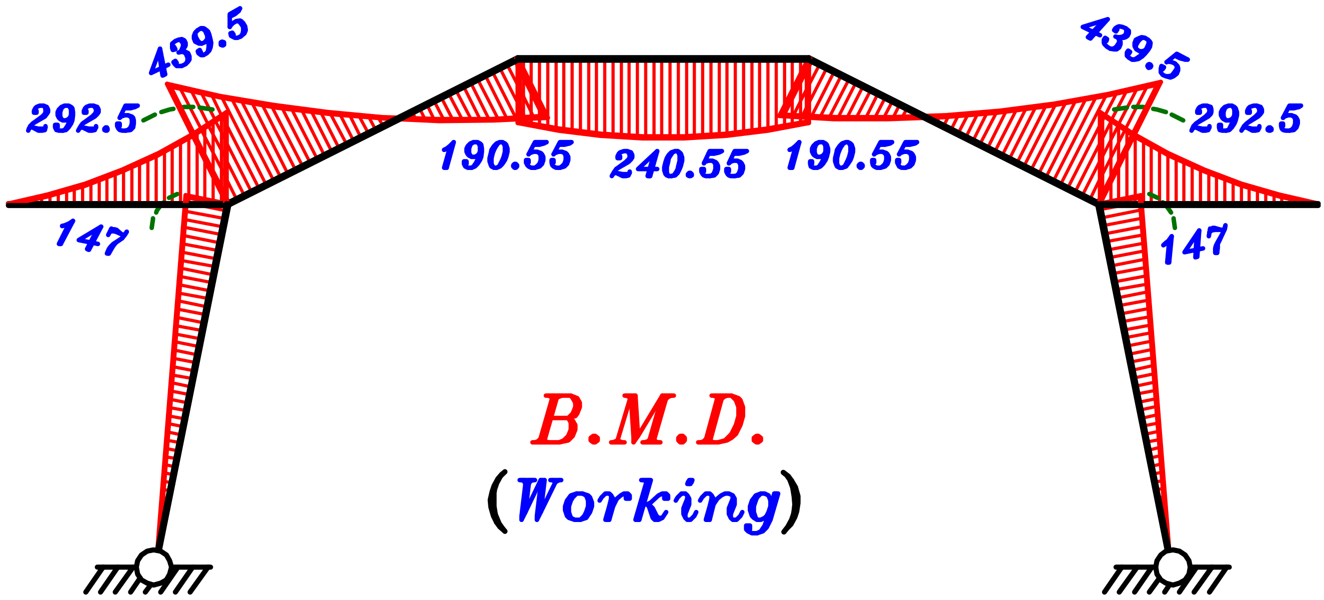




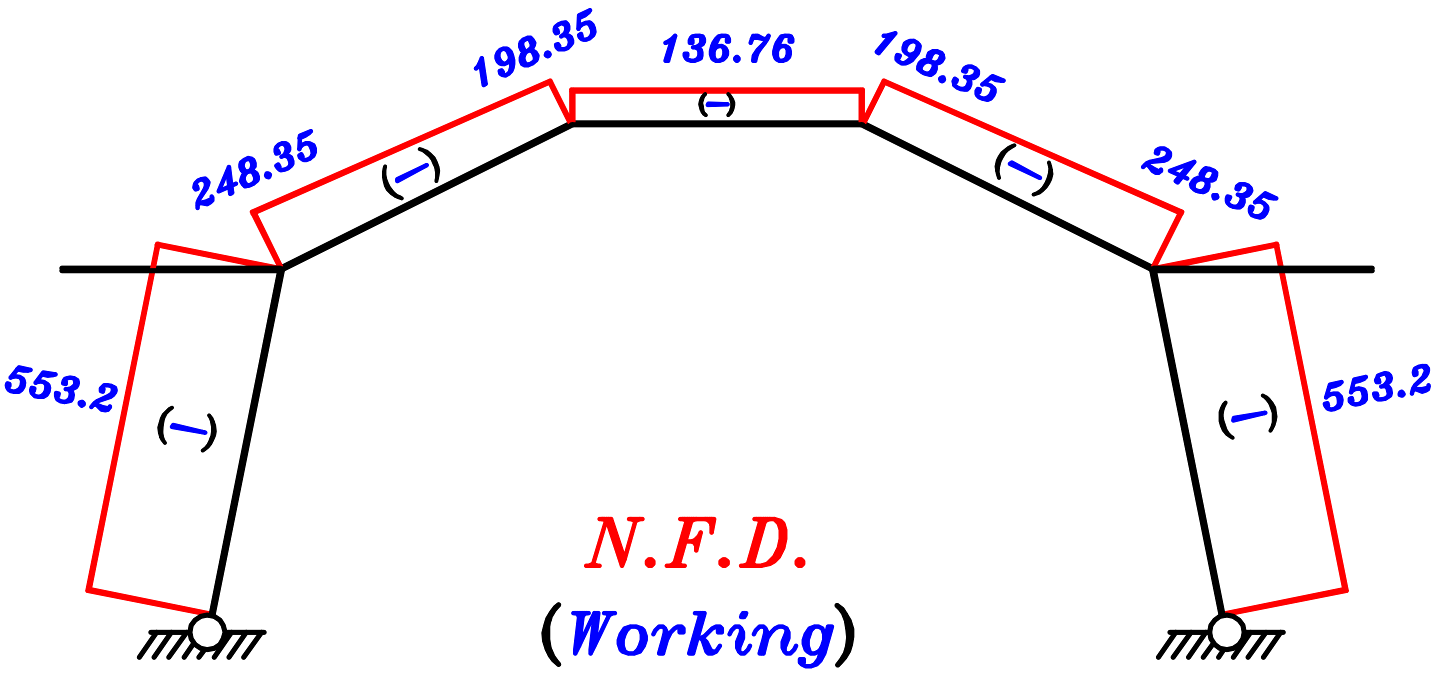




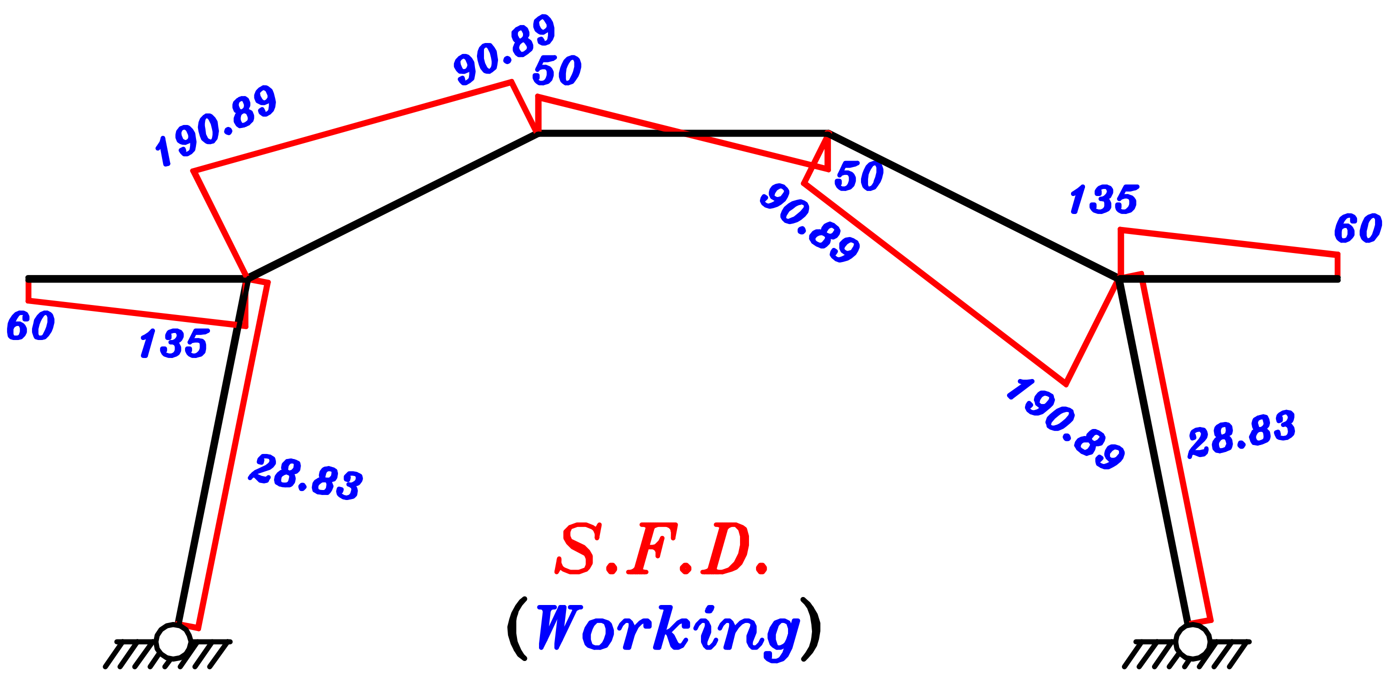
Statical System



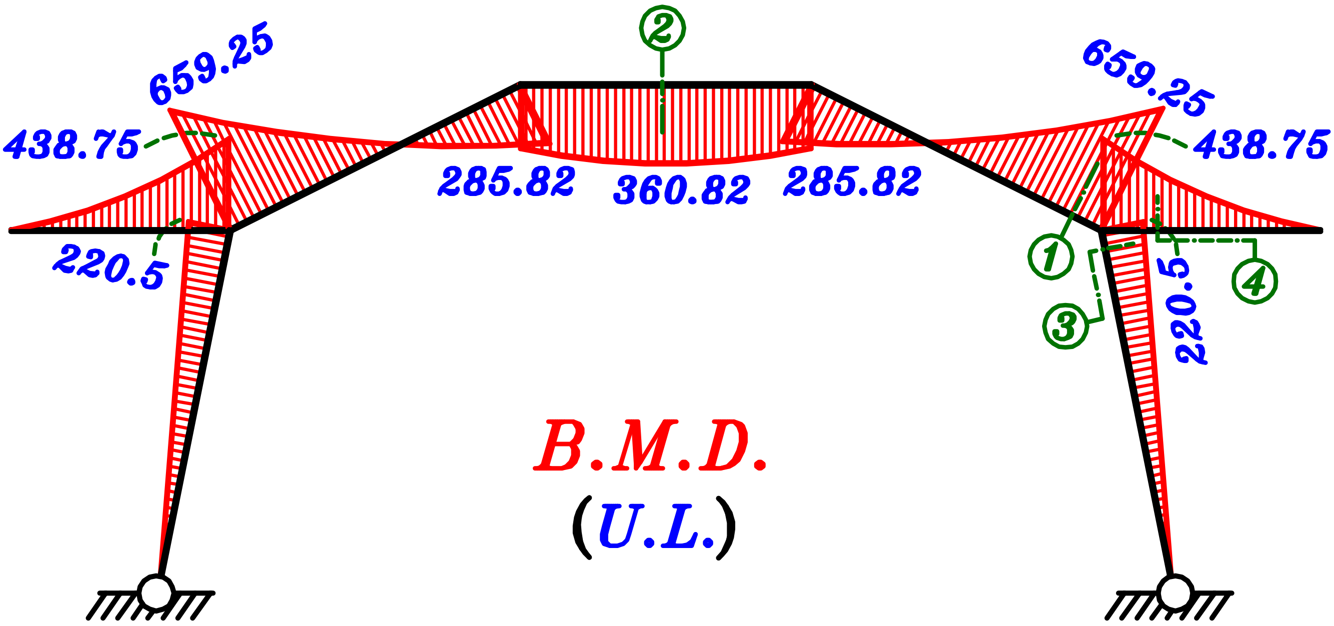
B.M.D.
(Working)



N.F.D.
(Working)

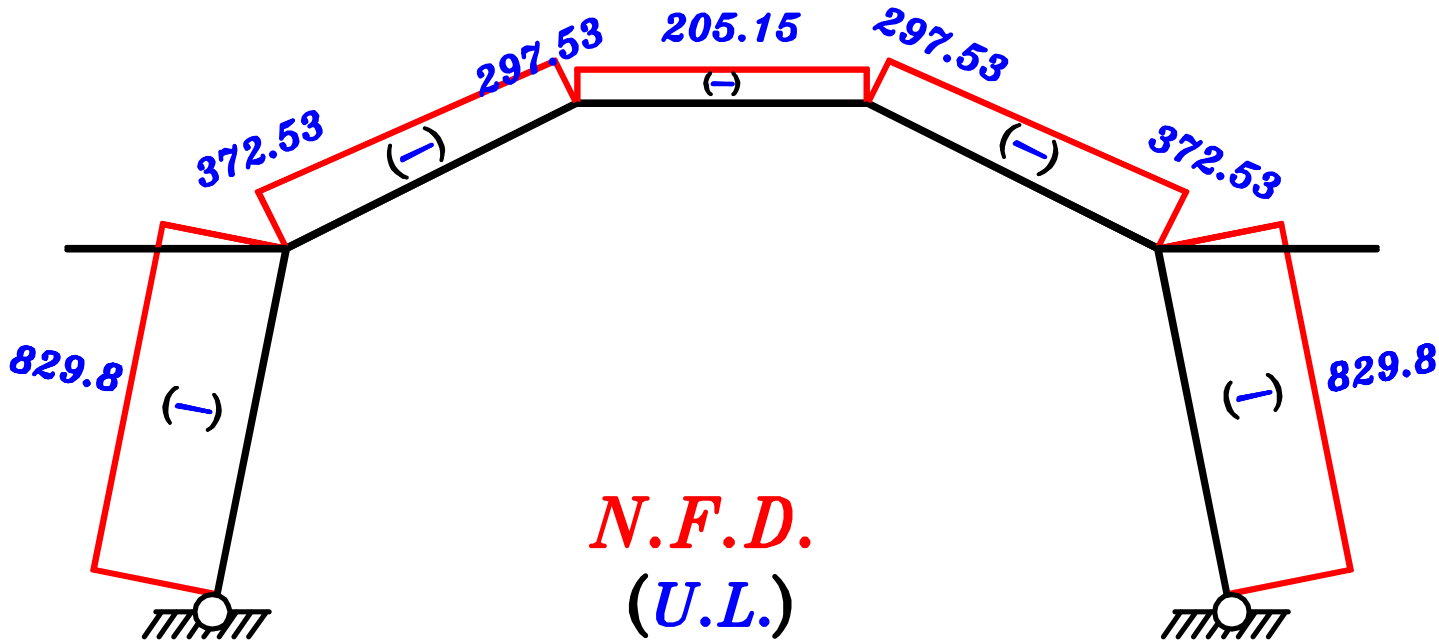


S.F.D.
(Working)



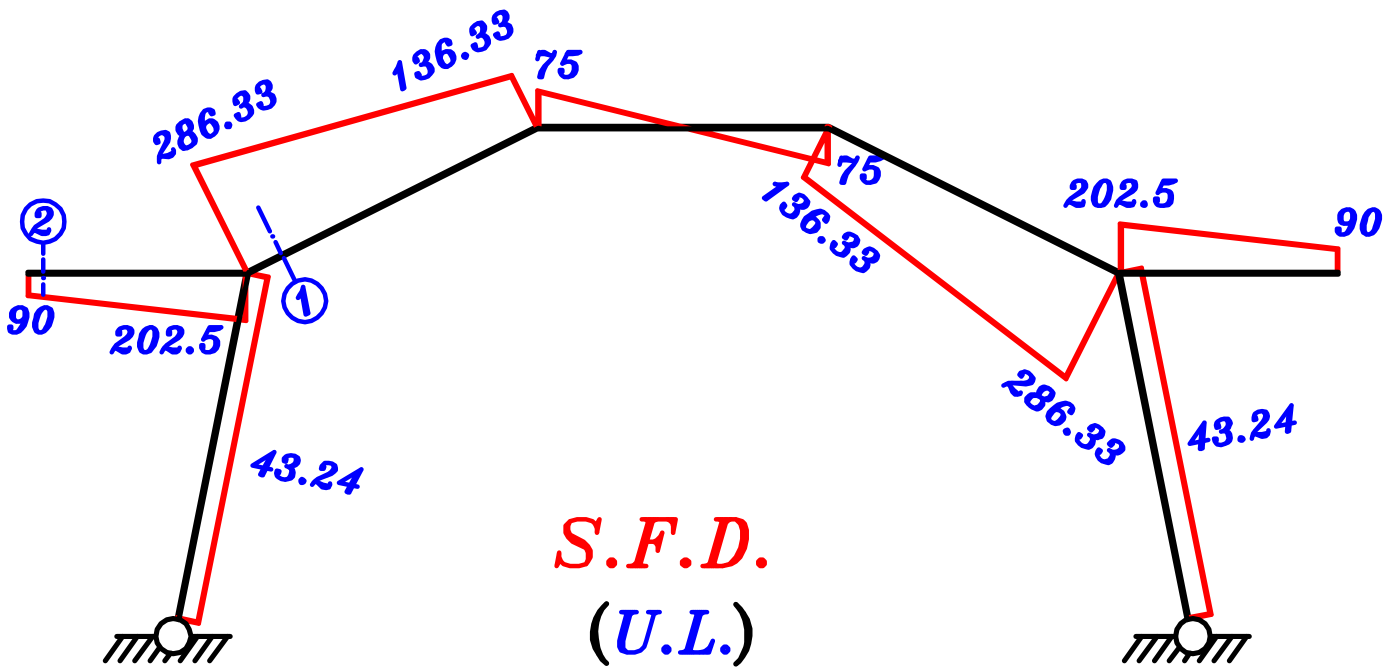
B.M.D.

(U.L.)



N.F.D.

(U.L.)



S.F.D.

(U.L.)

Design of Sections.

Sec. ①

$M = 659.25$ kN.m , $P = 372.53$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{659.25*10^6}{25*350}} = 960.7 \text{ mm} \qquad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(960.7) = (1056.77 \rightarrow 1248.9) \text{ mm}$$

Take $d = 1100$ mm , $t = 1100 + 100 = 1200$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{372.53*10^3}{25*350*1200} = 0.035 < 0.04 \quad \therefore$ (neglect P)

$\therefore$ Take $d = d_o = 960.7$ mm

$\therefore$ Take $\boxed{d = 1000 \text{ mm}}$, $\boxed{t = 1100 \text{ mm}}$

$\because \; C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{659.25*10^6}{0.780*360*1000} = 2347.75 \text{ mm}^2$$

Check $A_{s_{min.}}$

$A_{s_{req.}} = 2347.75$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225*\frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225*\frac{\sqrt{25}}{360}\right) 350*950 = 1039 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2347.75$ mm^2 10ϕ18

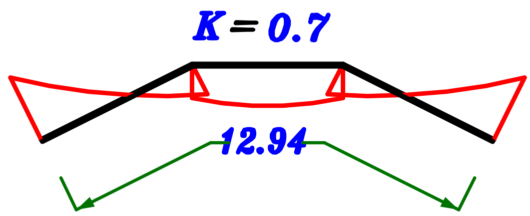
$$\therefore n = \frac{b-25}{\phi+25} = \frac{350-25}{18+25} = 7.56 = 7.0 \text{ bars}$$

Sec. ② $M = 360.82\ kN.m\ ,\ P = 205.15\ kN\ ,\ b = 350\ mm$

$d = 950\ mm$ (the same depth of Sec. ①)

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{205.15 * 10^3}{25 * 350 * 1000} = 0.0213 < 0.04 \quad \therefore$ (neglect P)

$\therefore$ The sec. will be T-Sec.



$$B = \begin{cases} C.L.-C.L. = Spacing = 5.0\ m = 5000\ mm \\ 16\ t_s + b = 16 * 120 + 350 = 2270\ mm \\ K\dfrac{L}{5} + b = 0.7 * \dfrac{12940}{5} + 350 = 2161.6\ mm \end{cases}$$

$\boxed{B = 2161.6\ mm}$

$$\therefore d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, B}} \quad \therefore 1000 = C_1 \sqrt{\frac{360.82 * 10^6}{25 * 2161.6}} \rightarrow C_1 = 12.24 \rightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{360.82 * 10^6}{0.826 * 360 * 1000} = 1213.41\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1213.41\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1000 = 1039.75\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1213.41\ mm^2$ $\quad 5\ \phi 18$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (0.1 \rightarrow 0.2)\ 1371$ $\quad 2\ \phi 12$

Sec. ③ R-Sec. $M = 220.5$ kN.m , $P = 829.8$ kN

$$d_o = 3.5\sqrt{\frac{220.5 * 10^6}{25 * 350}} = 555.6 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(555.6) = (611.17 \rightarrow 722.3) \text{ mm}$$

$\therefore$ Take $\boxed{d = 650 \text{ mm}}$, $\boxed{t = 700 \text{ mm}}$

$$\because \boxed{t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)} = 1100 \text{ mm}}$$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{829.8 * 10^3}{25 * 350 * 1100} = 0.086 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{220.5}{829.8} = 0.265 \text{ m} \quad \therefore \frac{e}{t} = \frac{0.265}{1.1} = 0.24 < 0.5 \xrightarrow{Use} \text{I.D.}$$

$\therefore$ Use Interaction Diagram

$$\zeta = \frac{1100 - 200}{1100} = 0.818 = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page 4-24}$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{829.8 * 10^3}{25 * 350 * 1100} = 0.086 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{220.5 * 10^6}{25 * 350 * 1100^2} = 0.021 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s' = \mu * b * t = 2.5 * 10^{-3} * 350 * 1100 = 962.5 \text{ mm}^2$$

$$- \text{ Check } A_{s\,min.} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 350 * 1100 = 3080 \text{ mm}^2$$

$$A_{s\,Total} = A_s + A_s' = 2 * 962.5 = 1925 \text{ mm}^2 \quad \therefore A_{s\,Total} < A_{s\,min.}$$

$$\therefore \text{take } A_s = A_s' = \frac{A_{s\,min.}}{2} = \frac{3080}{2} = 1540 \text{ mm}^2 \quad 7\phi18$$

Sec. ④ $M_{U.L.} = 438.7$ kN.m , R−Sec., $b = 350$ mm

$d = 600$ mm (As given in Data.)

$\because d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 600 = C_1 \sqrt{\dfrac{438.7 * 10^6}{25 * 350}} \longrightarrow C_1 = 2.68 < 2.78$

∴ We have to increase Dimensions

OR use $A_{s`}$ using First Principles.

OR use $A_{s`}$ using I.D.

Use $A_{s`}$ using First Principles.

$a_{max} = 0.8 \left(\frac{2}{3}\right) \left[\dfrac{600}{600 + (F_y \backslash \gamma_s)}\right] * d = 0.35\, d = 0.35 * 600 = 210$ mm

$M_{U.L._{max}} = \frac{2}{3} \frac{F_{cu}}{\gamma_c} a_{max}\, b \left(d - \frac{a_{max}}{2}\right) = \frac{2}{3}\left(\frac{25}{1.5}\right)(210)(350)\left(600 - \frac{210}{2}\right) = 404250000$

$= 404.25$ kN.m

− Get $\triangle M = M_{U.L.} - M_{U.L._{max.}} = 438.7 - 404.25 = 34.45$ kN.m

− Get $A_{s`}$ From $\triangle M = A_{s`} \dfrac{F_y}{\gamma_s} (d - d`)$

$\therefore 34.45 * 10^6 = A_{s`} \left(\frac{360}{1.15}\right)(600 - 50) \longrightarrow A_{s`} = 200\ mm^2$

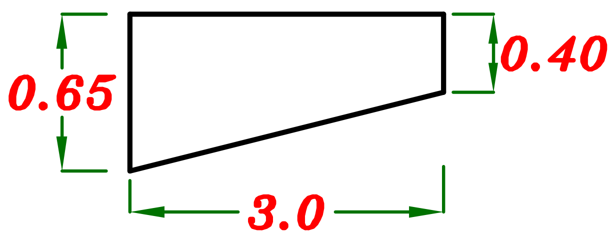
2ϕ12

$\mu_{max.} = 5 * 10^{-4} F_{cu} = 5 * 10^{-4} * 25 = 0.0125$ From old Tables Page (6)

$\therefore A_s = \mu_{max.}\, b\, d + A_{s`} = 0.0125(350)(600) + 200 = 2825\ mm^2$

12ϕ18

− Check $\dfrac{A_{s`}}{A_s} = \dfrac{200}{2825} = 0.07 < 0.40 \therefore$ o.k.



Check Shear.

- **Allowable shear stress.**

$$- \; q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{25}{1.5}} = 0.98 \quad N\backslash mm^2$$

$$- \; q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{25}{1.5}} = 2.85 \quad N\backslash mm^2$$

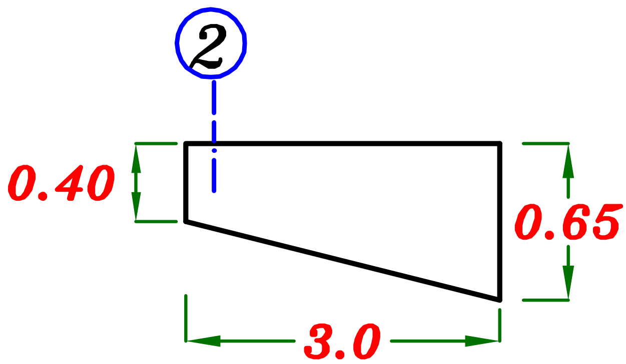
Sec. ① $\quad Q = 286.33 \; kN$

$$\therefore \; q_U = \frac{Q}{b\,d} = \frac{286.33 * 10^3}{350 * 1000} = 0.818 \; N\backslash mm^2$$

$$q_U < q_{cu} \longrightarrow \text{Use min. stirrups} \quad \boxed{5\phi 8 \backslash m`}$$

Sec. ② $\quad Q = 90 \quad kN$

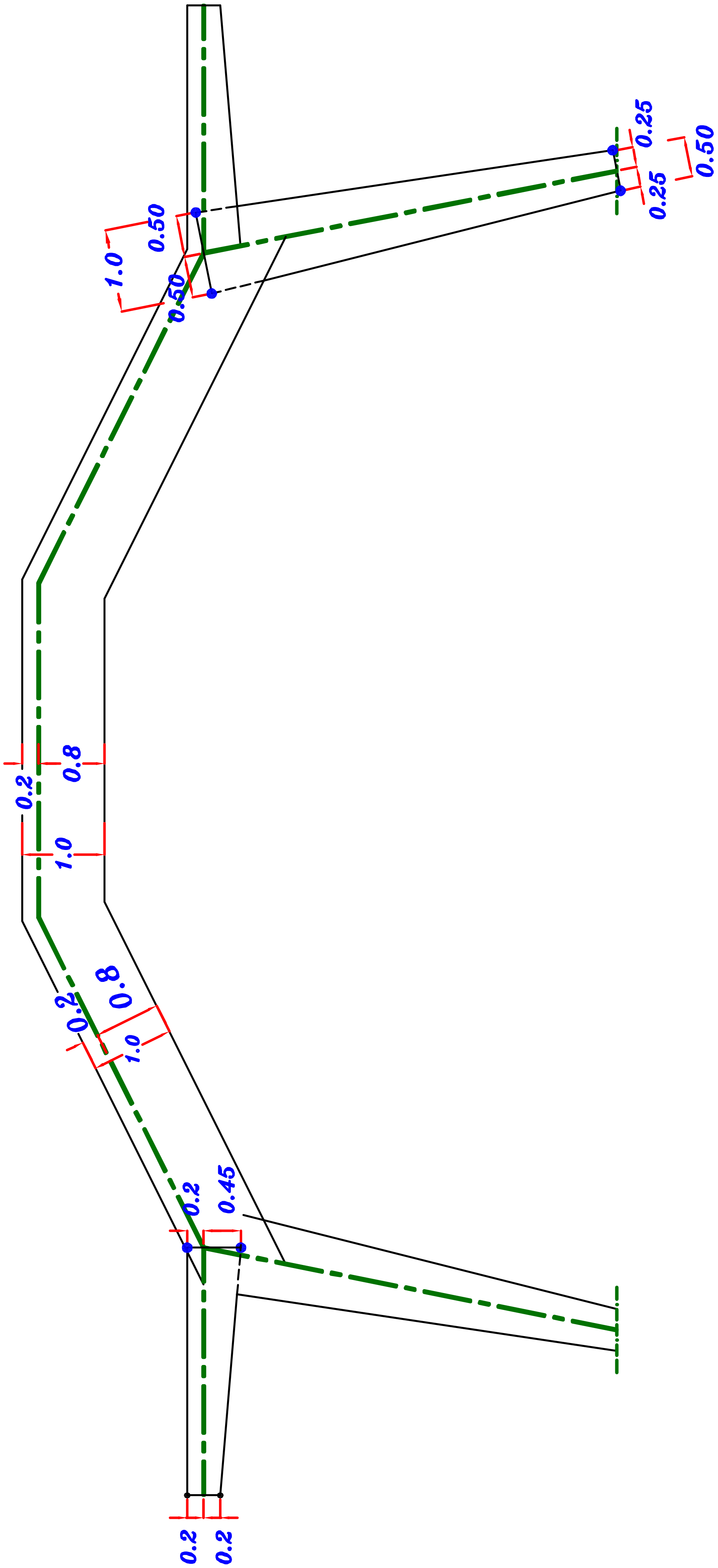
$\tan\beta = Zero$



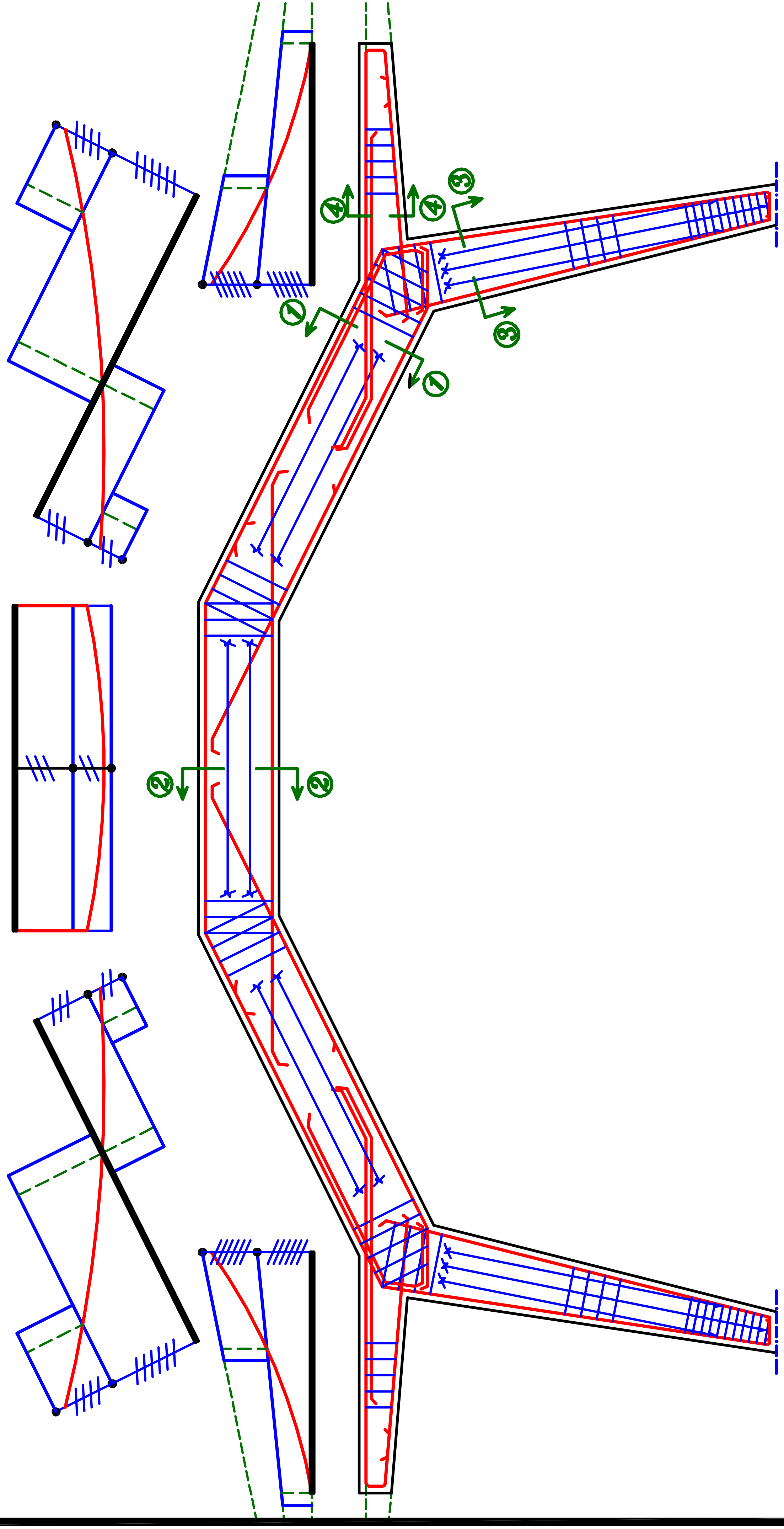
$$\therefore \; \text{Actual shear stress.} = q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\;d^2}$$

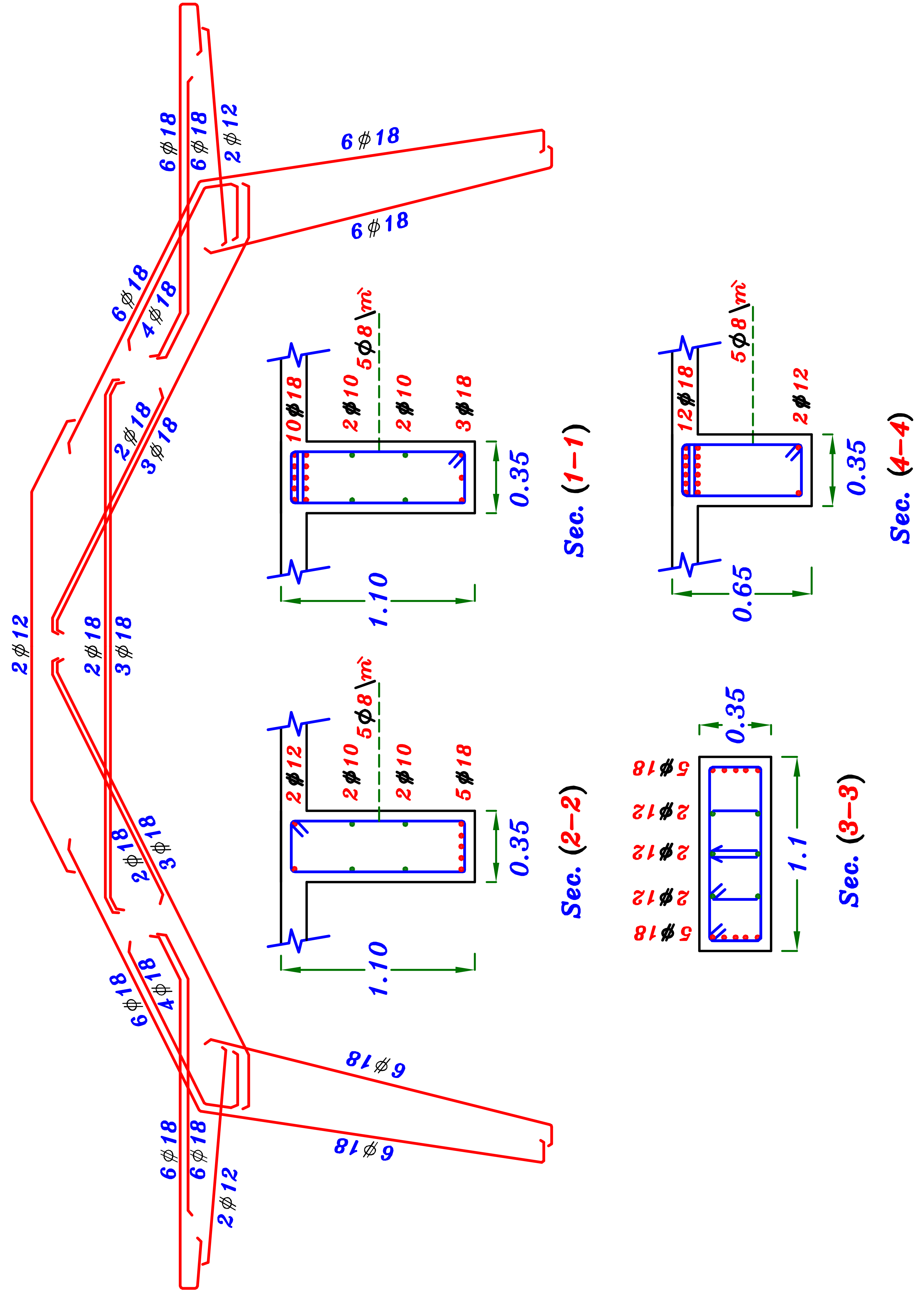
$$= \frac{90 * 10^3}{350 * 350} - ZERO = 0.73 \quad N\backslash mm^2$$

$$\therefore \; q_U < q_{cu} \longrightarrow \text{Use min. stirrups} \quad \boxed{5\phi 8 \backslash m`}$$


0.25
0.50
0.25
0.50
1.0
0.50
0.2
0.8
1.0
0.2
0.8
1.0
0.2
0.45
0.2
0.2

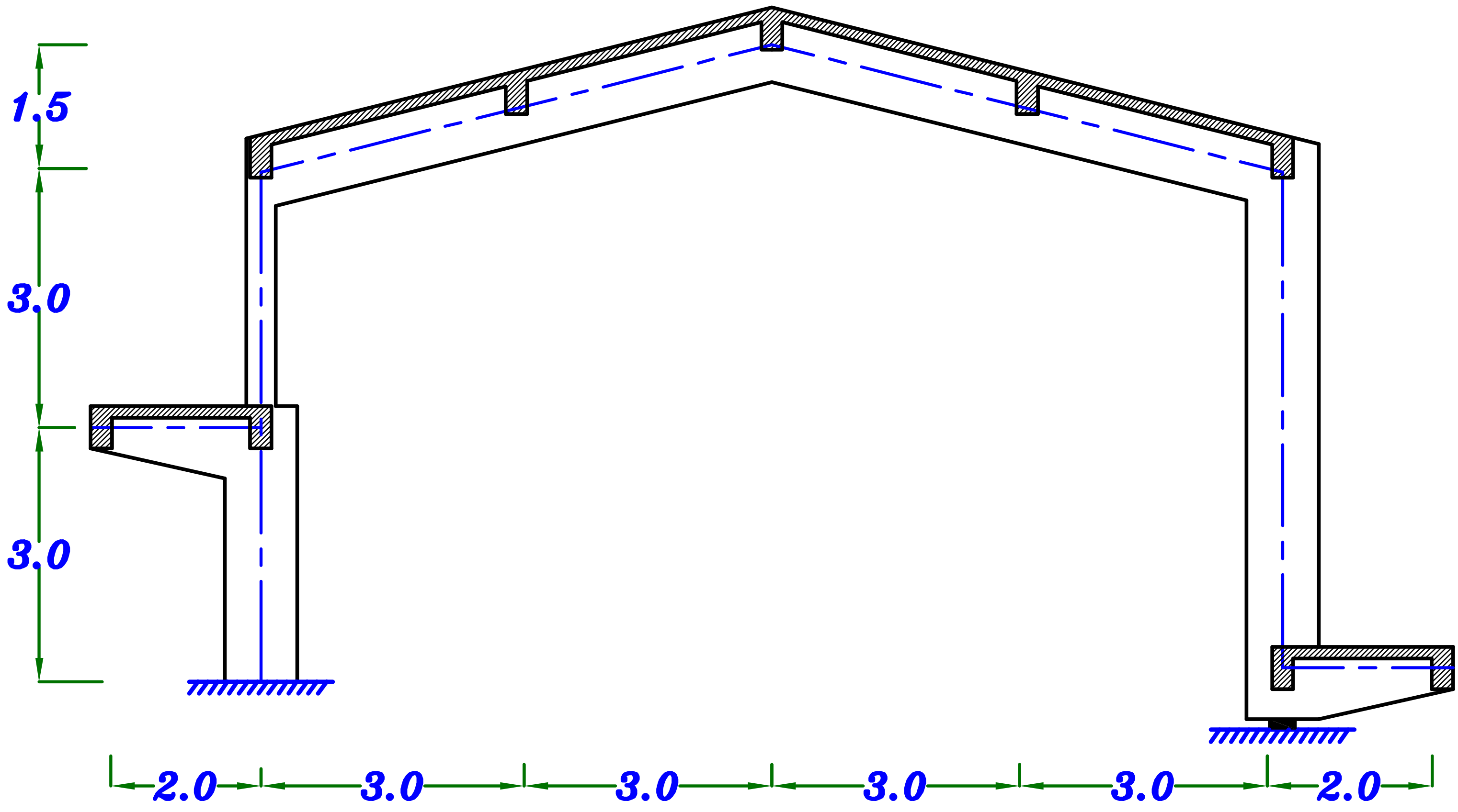
R.F.T. of the Frame.

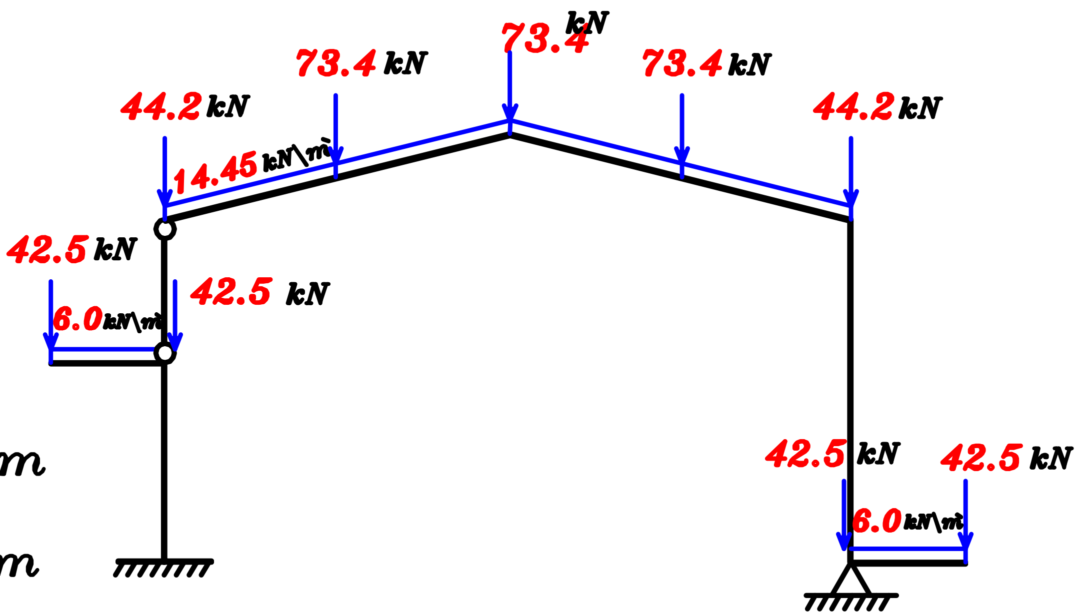




6ϕ18
6ϕ18
2ϕ12
6ϕ18
6ϕ18
6ϕ18
4ϕ18
2ϕ18
3ϕ18
2ϕ12
2ϕ18
3ϕ18
2ϕ18
3ϕ18
6ϕ18
4ϕ18
6ϕ18
6ϕ18
6ϕ18
6ϕ18
2ϕ12
10ϕ18
2ϕ10
2ϕ10
3ϕ18
5ϕ8\m`
0.35
1.10
Sec. (1-1)
2ϕ12
2ϕ10
2ϕ10
5ϕ18
5ϕ8\m`
0.35
1.10
Sec. (2-2)
12ϕ18
2ϕ12
5ϕ8\m`
0.35
0.65
Sec. (4-4)
5ϕ18
2ϕ12
2ϕ12
2ϕ12
5ϕ18
0.35
1.1
Sec. (3-3)

Example.





Data.

$F_{cu} / F_y = 25/360$

$t_s = 140$ mm

Spacing = 6.0 m

$b_{(Beams)} = 250$ mm

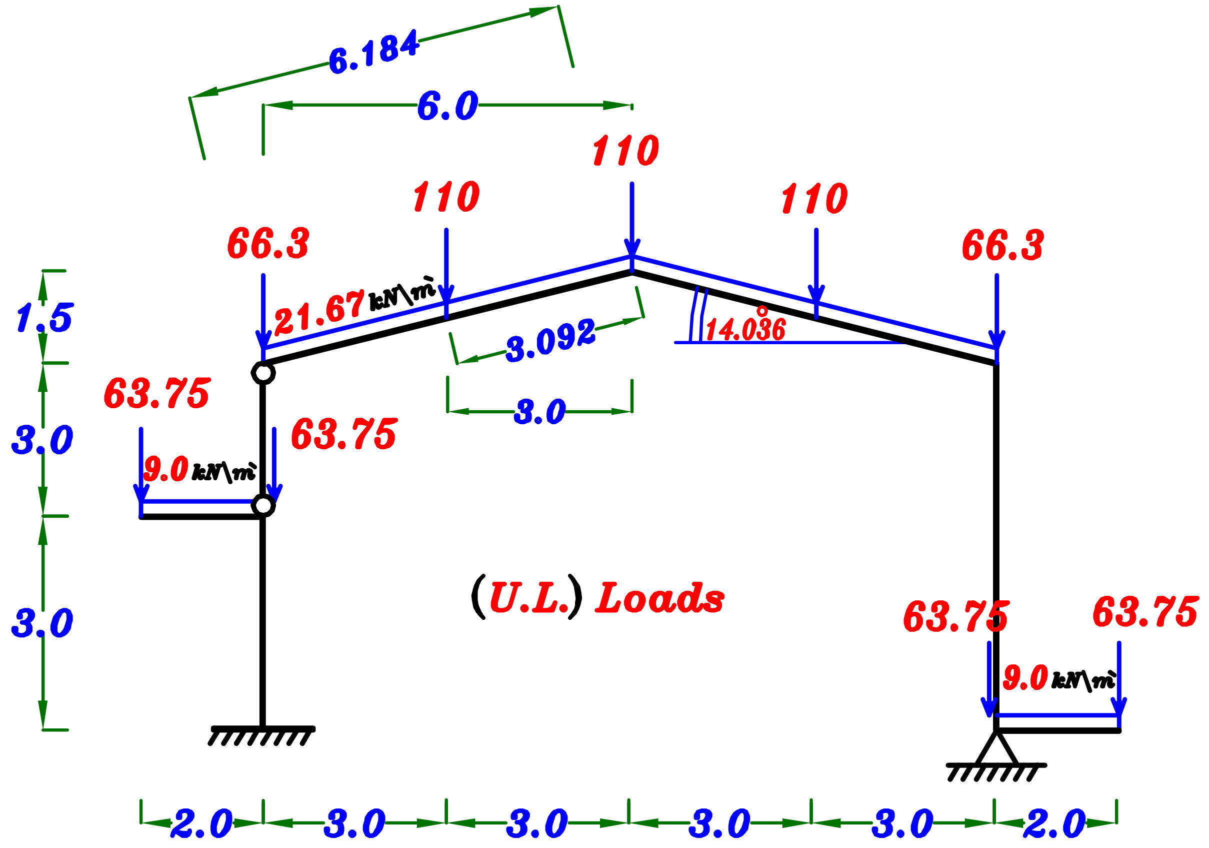
$b_{(Frames)} = 350$ mm

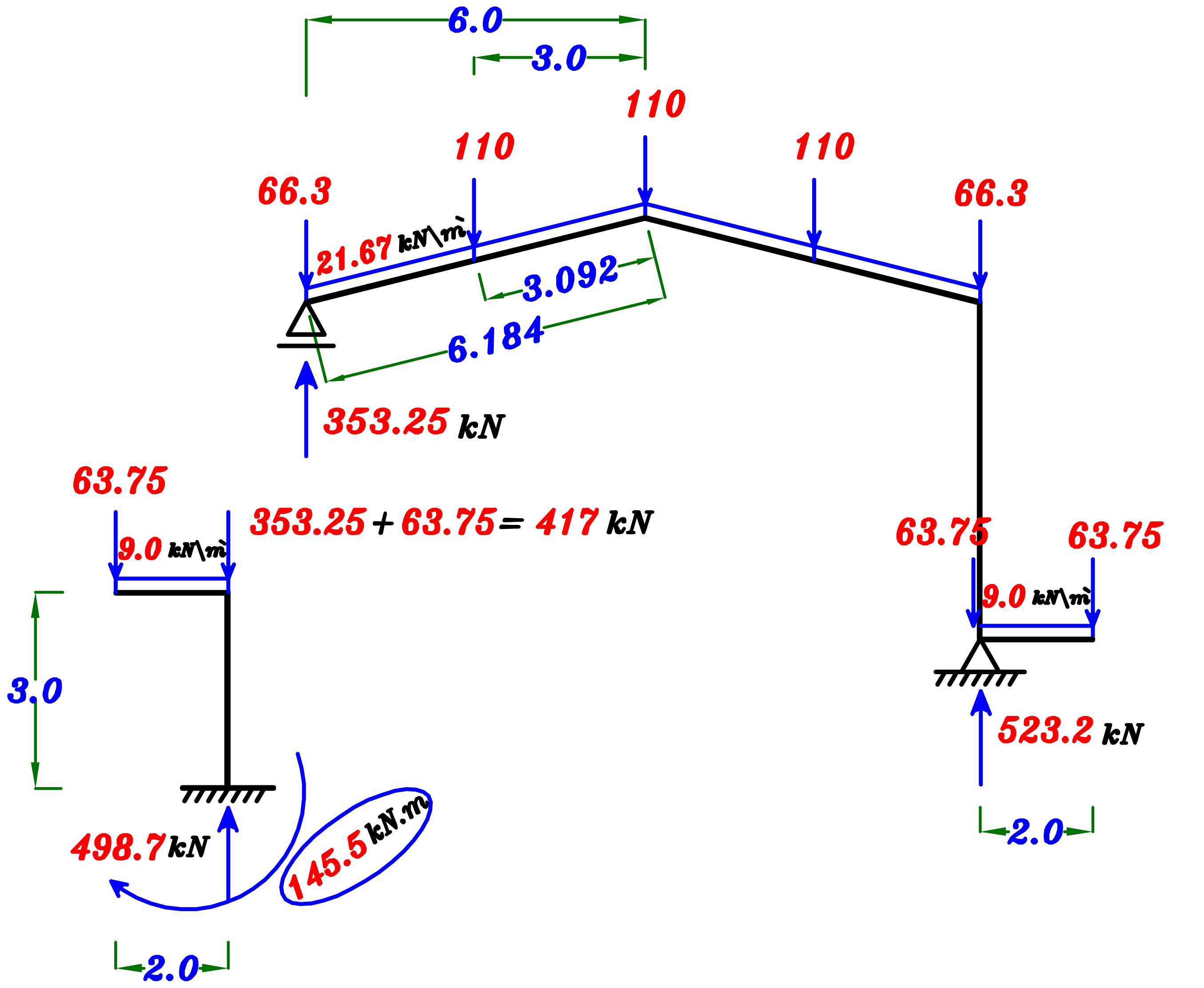
Req.

① Draw I.F.D. due to U.L. Loads.

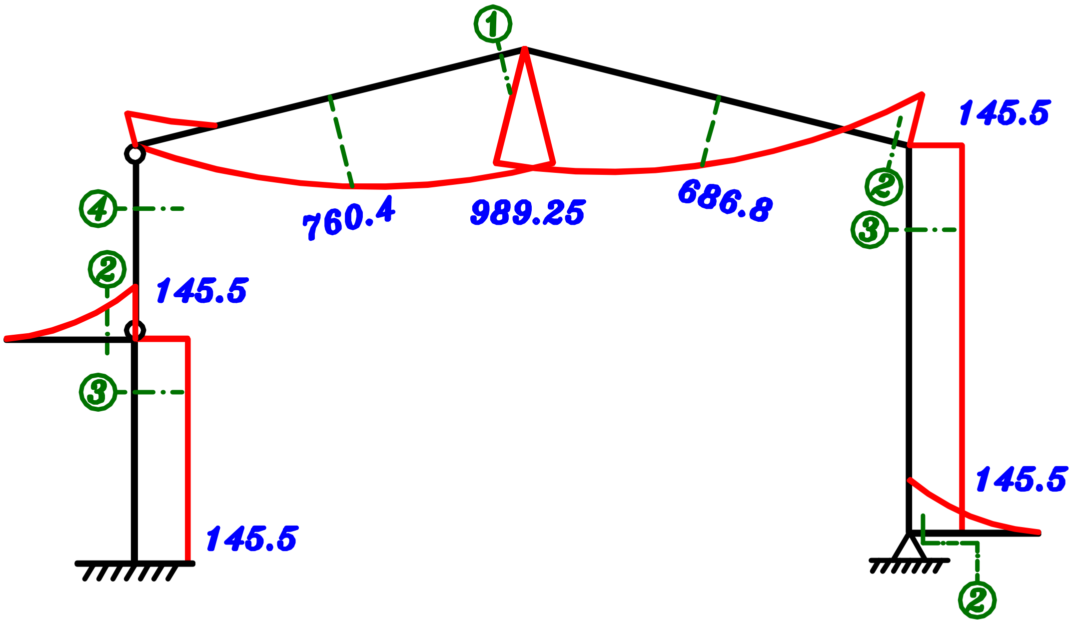
② Design the critical sections of the Frame using U.L.D.M.

③ Draw details of RFT. to scale 1:50 in elevation & scale 1:20 cross-sec. making curtailment of steel using Moment of Resistance.

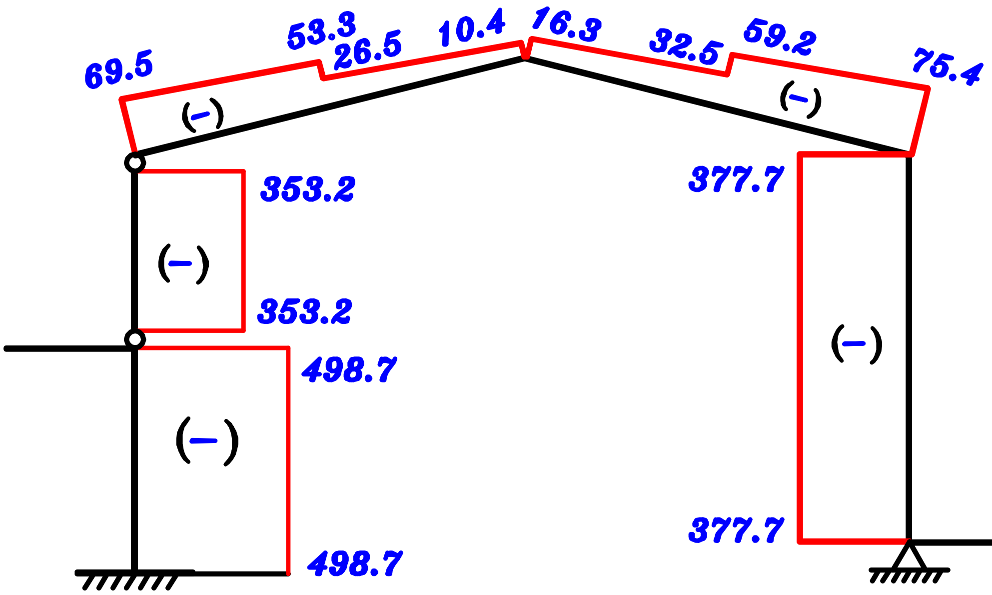

6.184
6.0
110
110
110
66.3
66.3
21.67 kN\m`
3.092
14.036°
1.5
3.0
3.0
3.0
63.75
63.75
9.0 kN\m`
(U.L.) Loads
63.75
63.75
9.0 kN\m`
2.0
3.0
3.0
3.0
3.0
2.0


6.0
3.0
110
110
110
66.3
66.3
21.67 kN\m`
3.092
6.184
353.25 kN
63.75
353.25 + 63.75 = 417 kN
9.0 kN\m`
3.0
498.7 kN
145.5 kN.m
2.0
63.75
63.75
9.0 kN\m`
523.2 kN
2.0

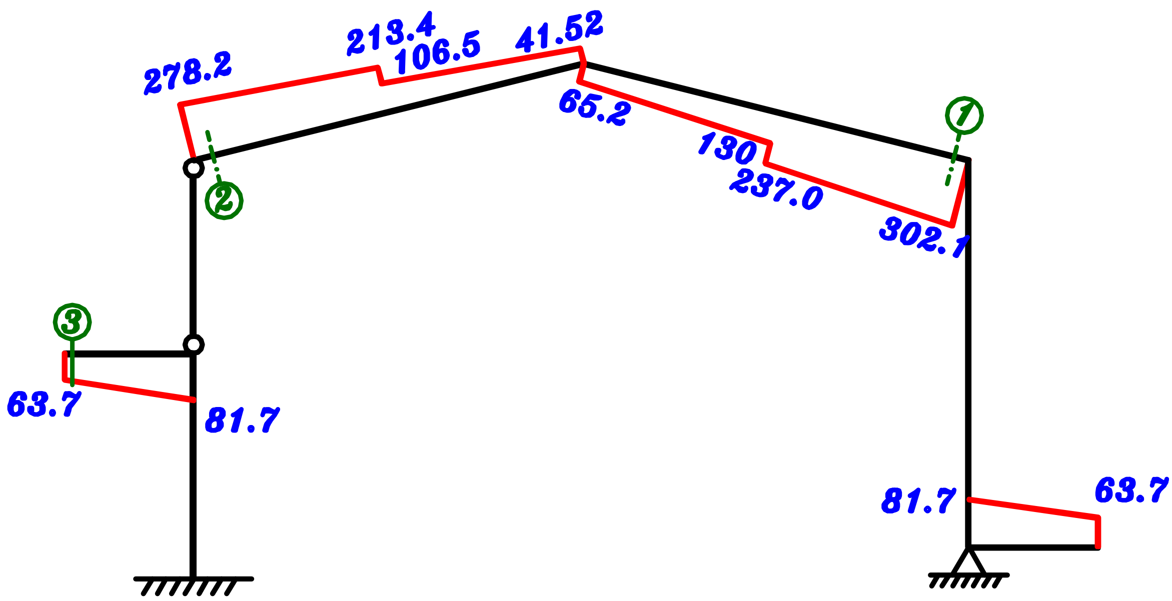
1 I.F.D. For the Frame.



B.M.D.
(U.L.)



N.F.D.
(U.L.)



S.F.D.
(U.L.)

Design of Sections.

Sec. 1

$M = 989.25$ kN.m , $P = 10.4$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{989.25 * 10^6}{25 * 350}} = 1176.8 \text{ mm} \quad (\text{as R-Sec.})$$

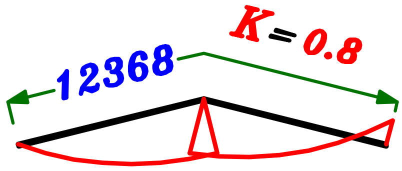
$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)(1176.8) = (1294.5 \rightarrow 1529) \text{ mm}$$

Take $d = 1300$ mm , $t = 1300 + 100 = 1400$ mm

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{10.4 * 10^3}{25 * 350 * 1400} = 0.00085 < 0.04 \;\therefore$ (neglect P)

The sec. will be T-sec.

$$B = \left\{ \begin{array}{l} C.L.-C.L. = \text{Spacing} = 6.0 \text{ m} = 6000 \text{ mm} \\ 16\, t_s + b = 16 * 140 + 350 = 2590 \text{ mm} \\ K\dfrac{L}{5} + b = 0.8 * \dfrac{12368}{5} + 350 = 2328 \text{ mm} \end{array} \right\}$$



$$\boxed{B = 2328 \text{ mm}}$$

Take $C_1 = 6.0 \longrightarrow J = 0.826$

$$\therefore d = 6.0\sqrt{\frac{989.25 * 10^6}{25 * 2328}} = 782.2 \text{ mm}$$

$\therefore$ Take $\boxed{d = 800 \text{ mm}}$, $\boxed{t = 850 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{989.25 * 10^6}{0.826 * 360 * 782.2} = 4253 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 4253 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 800 = 875 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \;\therefore$ Take $A_s = A_{s_{req.}} = 4253 \text{ mm}^2$ $\quad 12\phi 22$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{22 + 25} = 6.91 = 6.0 \text{ bars}$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 4253$ $\quad 4\phi 12$

Sec. ② $M = 145.5$ kN.m , $P = 75.4$ kN , $b = 350$ mm

$d = 800$ mm (the same depth of Sec. ①)

Check $\frac{P}{F_{cu}\, b\, t} = \frac{75.4 * 10^3}{25 * 350 * 850} = 0.010 < 0.04 \therefore$ (neglect P)

$\therefore d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 800 = C_1 \sqrt{\frac{145.5 * 10^6}{25 * 350}} \rightarrow C_1 = 6.20 \rightarrow J = 0.826$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{145.5 * 10^6}{0.826 * 360 * 800} = 611 \text{ mm}^2$$

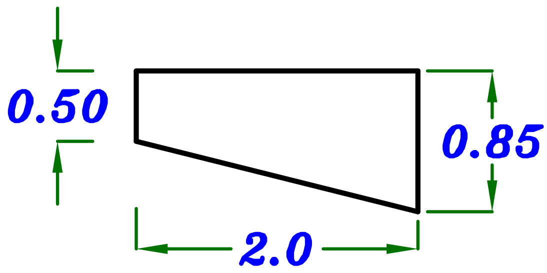
Check $A_{s_{min.}}$ $A_{s_{req.}} = 611 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 800 = 875 \text{ mm}^2$$

$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 800 = 875 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 611 = 794.3 \text{ mm}^2 \end{array}\right\} \text{الأقل} = 794.3 \text{ mm}^2 \quad 3 \phi 22$$

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{850}{2} = 425 = 450 \text{ mm} \\ t_b = \frac{\text{spacing}}{12} = \frac{6000}{12} = 500 \text{ mm} \\ t - \frac{L_c}{3} = 850 - \frac{2000}{3} = 183 \text{ mm} \end{array} \right\} Y = 500 \text{ mm}$$



Sec. ③   R–Sec. $M = 145.5$ kN.m , $P = 377.7$ kN

$$d_o = 3.5\sqrt{\frac{145.5 * 10^6}{25 * 350}} = 451.3 \text{ mm} \quad (\text{as R–Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(451.3) = (496.4 \rightarrow 586.7) \text{ mm}$$

$\therefore$ Take $\boxed{d = 500 \text{ mm}}$, $\boxed{t = 550 \text{ mm}}$

$\because$ $\boxed{t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)} = 850 \text{ mm}}$

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{377.7 * 10^3}{25 * 350 * 850} = 0.0507 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$e = \dfrac{M}{P} = \dfrac{145.5}{377.7} = 0.385$ m $\quad \therefore \dfrac{e}{t} = \dfrac{0.385}{0.85} = 0.45 < 0.5 \xrightarrow{Use}$ I.D.

$\therefore$ Use Interaction Diagram

$\zeta = \dfrac{850 - 100}{850} = 0.88 = 0.80 \xrightarrow{use}$ ECCS Design Aids Page 4–24

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{377.7 * 10^3}{25 * 350 * 850} = 0.0507 \\ \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{145.5 * 10^6}{25 * 350 * 850^2} = 0.023 \end{array}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 350 * 850 = 743 \text{ mm}^2$$

– Check $A_{s_{min}} = \dfrac{0.8}{100} * b * t = \dfrac{0.8}{100} * 350 * 850 = 2380 \text{ mm}^2$

$A_{s_{Total}} = A_s + A_s` = 2 * 743 = 1486 \text{ mm}^2 \quad \therefore A_{s_{Total}} < A_{s_{min}}$

$\therefore$ take $A_s = A_s` = \dfrac{A_{s_{min}}}{2} = \dfrac{2380}{2} = 1190 \text{ mm}^2$ $\quad$ 4 ϕ 22

Sec. ④ (350*400)

Axially Loaded Column. $P = 353.2$ kN

$\therefore P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$

$\therefore 353.2 * 10^3 = 0.35\,(350*400)(25) + 0.67\, A_s\,(360)$

$\therefore A_s = -\,3614 \ mm^2 = (-Ve)$ Value

$\therefore A_s = A_{s_{min.}} = \frac{0.8}{100} * 350 * 400 = 1120\ mm^2$ $10\phi12$

Check Shear.

- Allowable shear stress.

- $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{25}{1.5}} = 0.98\ N\backslash mm^2$

- $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{25}{1.5}} = 2.85\ N\backslash mm^2$

Sec. ① $Q = 302.1$ kN

$$q_U = \frac{Q}{b\,d} = \frac{302.1 * 10^3}{350 * 800} = 1.07\ N\backslash mm^2$$

$\therefore q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi8\backslash m'$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n\,A_s\,(F_y\backslash\gamma_s)}{b\ \ S}$

* Take $n = 2$, $\phi\, 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.07 - \frac{0.98}{2} = \frac{2 * 50.3\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 103.4\ mm > 100\ mm$$

$\therefore$ o.k.

$\therefore$ No. of stirrups$\backslash m' = \frac{1000}{S} = \frac{1000}{103.4} = 9.67 = 10$

$\therefore$ Use Stirrups $\boxed{10\phi8\backslash m'}$ 2 branches

Sec. ② $Q = 278.2$ kN

$$q_U = \frac{Q}{b\,d} = \frac{278.2 * 10^3}{350 * 800} = 0.993 \ N \backslash mm^2$$

$\therefore \ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi8 \backslash m'$

$$\therefore \text{Use} \quad q_s = q_u - \frac{q_{cu}}{2} = \frac{n\,A_s\,(F_y \backslash \gamma_s)}{b \;\; S}$$

* Take $n = 2$, $\phi\,8 \longrightarrow A_s = 50.3 \ mm^2$

$$0.993 - \frac{0.98}{2} = \frac{2 * 50.3\,(240 \backslash 1.15)}{350 * S} \longrightarrow S = 119.2 \ mm > 100 \ mm$$

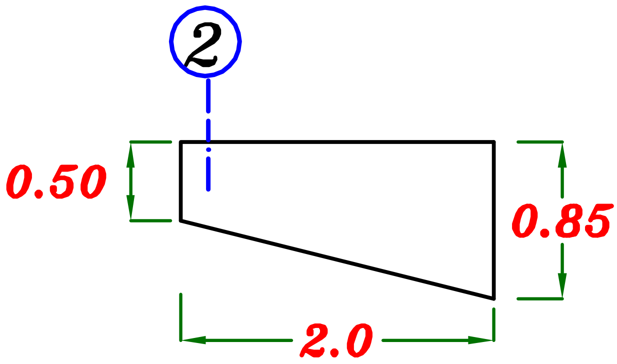
$\therefore$ o.k.

$$\therefore \text{No. of stirrups} \backslash m' = \frac{1000}{S} = \frac{1000}{119.2} = 8.39 = 9.0$$

$\therefore$ Use Stirrups $\boxed{9\phi8 \backslash m'}$ 2 branches

Sec. ③ $Q = 63.7$ kN

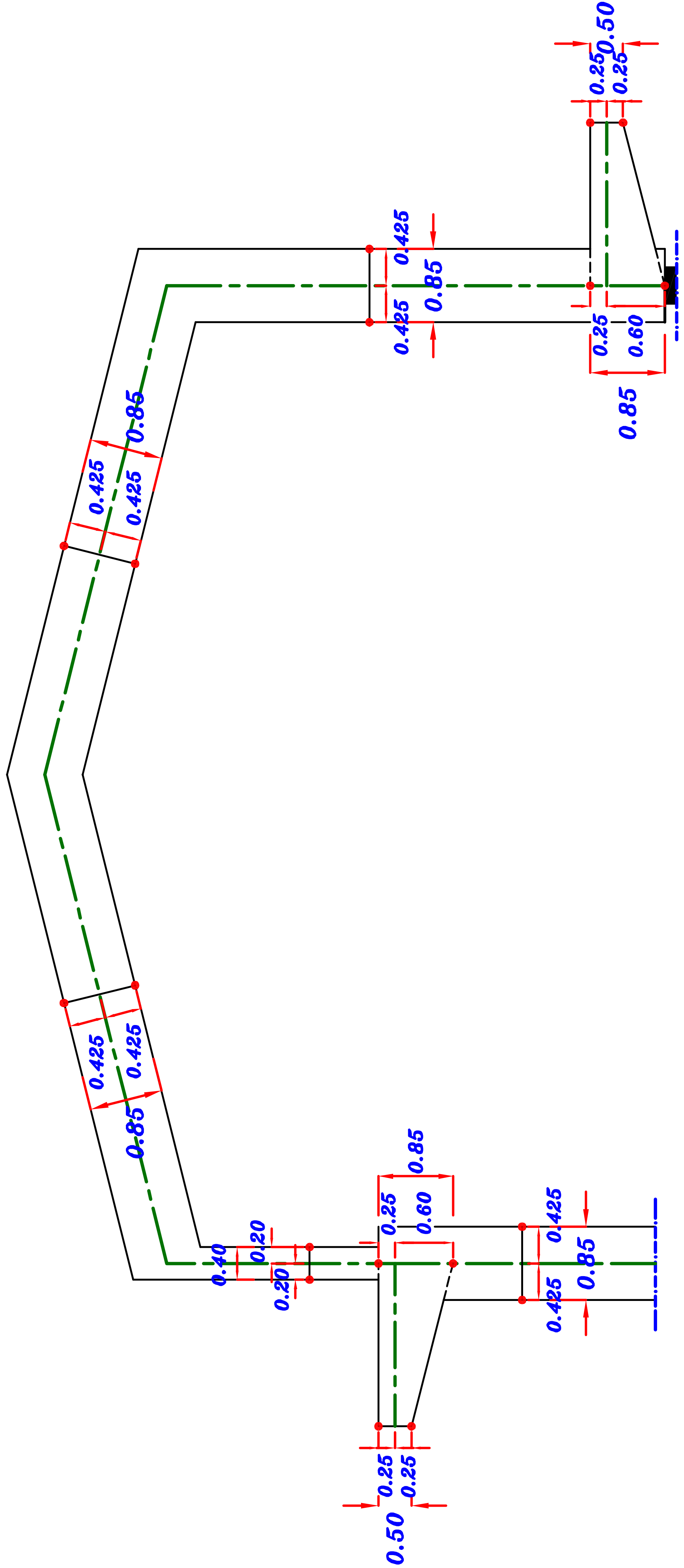
$\tan\beta = \text{Zero}$



$$\therefore \text{Actual shear stress.} = q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\;d^2}$$

$$= \frac{63.7 * 10^3}{350 * 450} - \text{ZERO} = 0.404 \ N \backslash mm^2$$

$\therefore \ q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi8 \backslash m'}$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0.25
0.425
0.85
0.425
0.25
0.60
0.85
0.85
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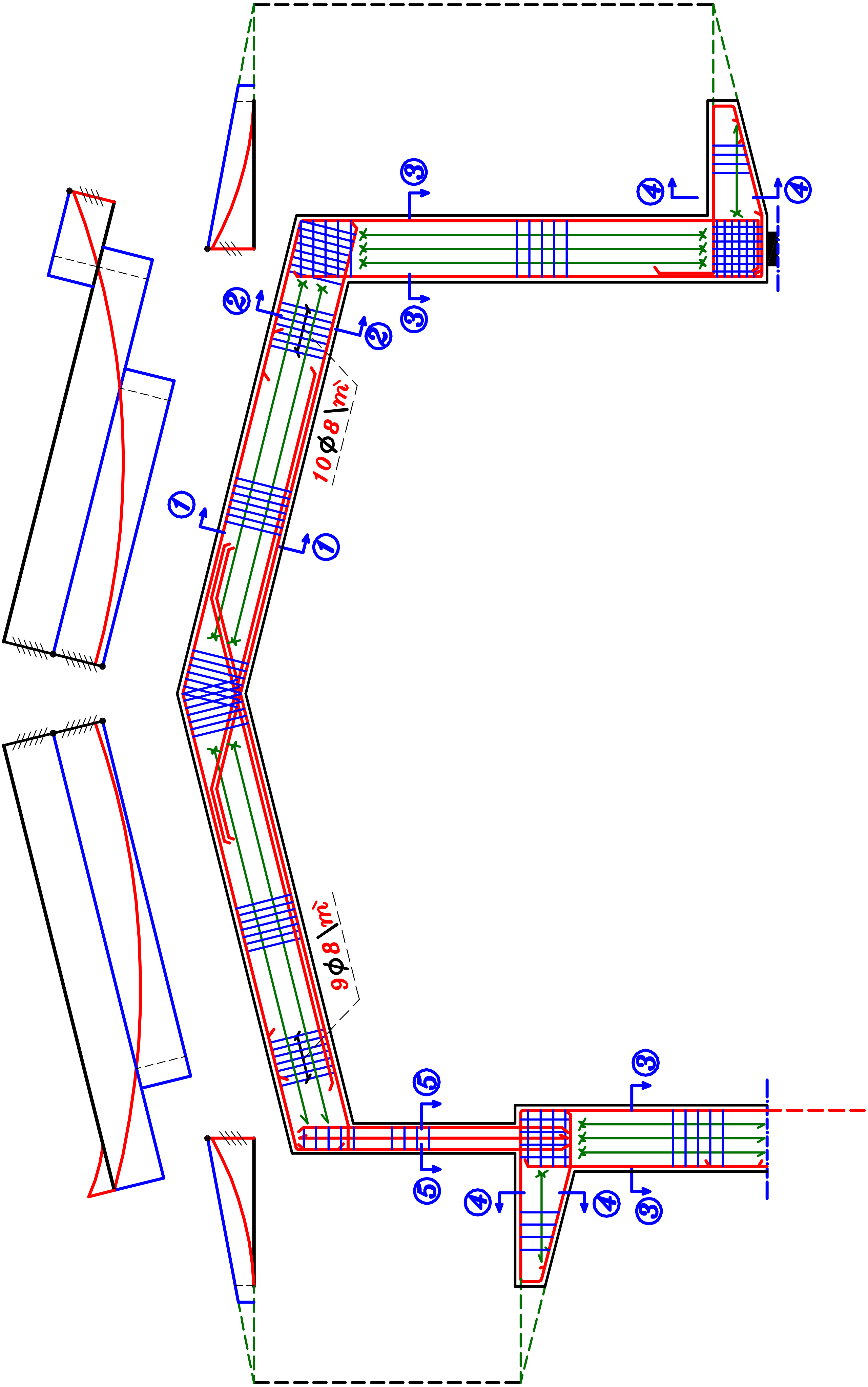
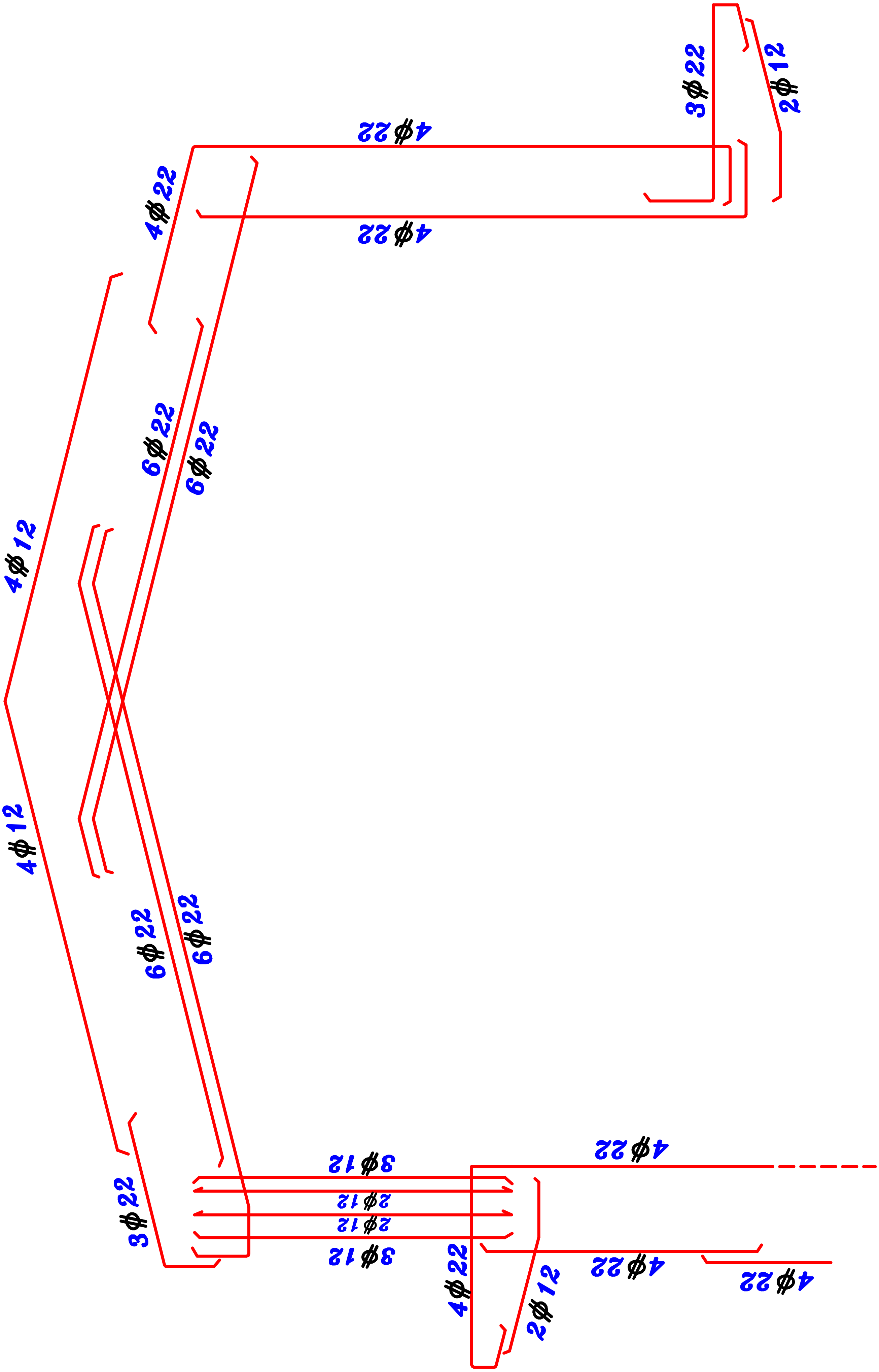
0.425
0.425
0.85
0.85
0.25
0.60
0.425
0.85
0.425
0.20
0.40
0.20
0.25
0.25
0.50

R.F.T. of the Frame.




3φ22
2φ12
4φ22
4φ22
4φ22
6φ22
6φ22
4φ12
4φ12
6φ22
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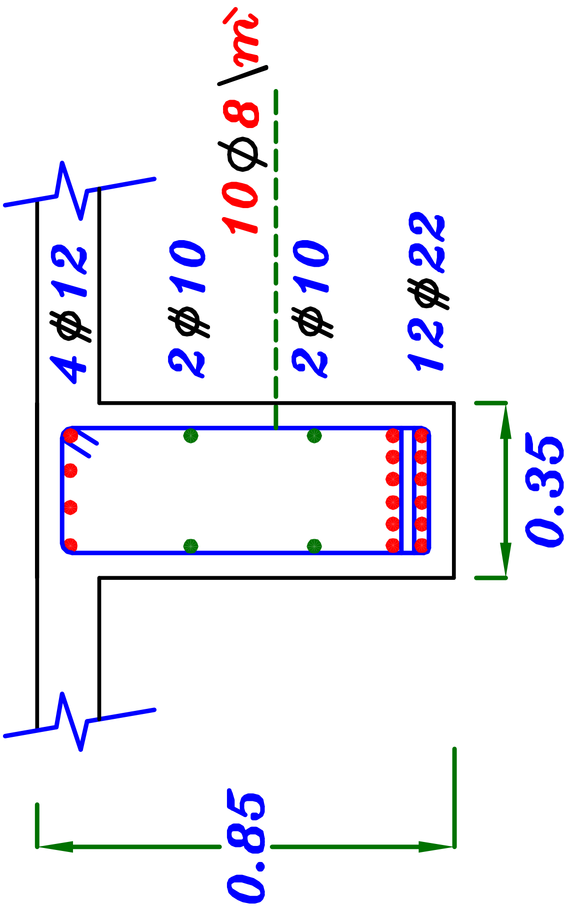
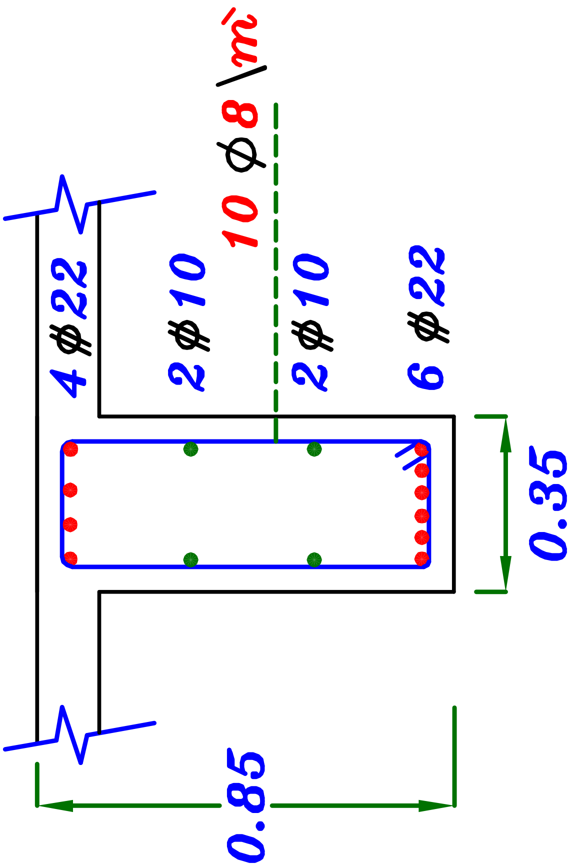
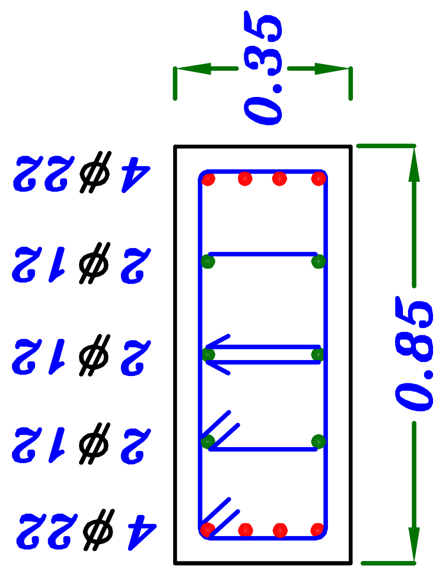
2φ12
4φ22
4φ22



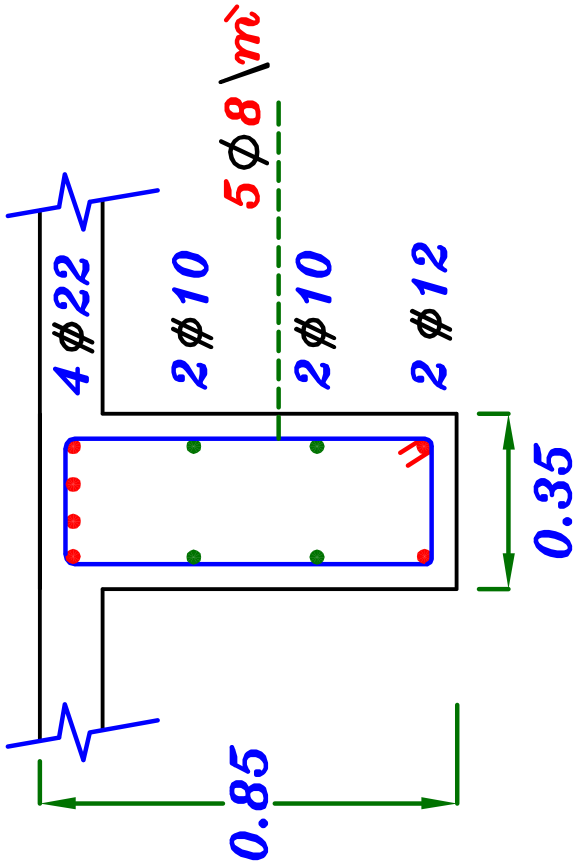
Sec. (1–1)



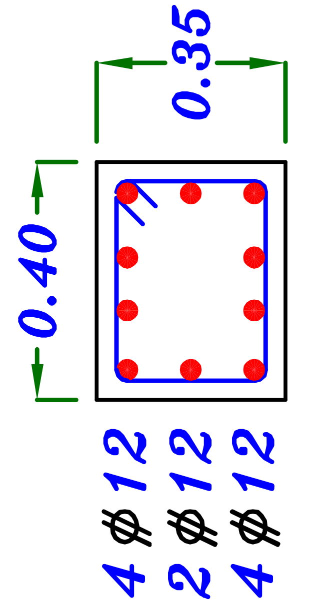
Sec. (2–2)



Sec. (3–3)

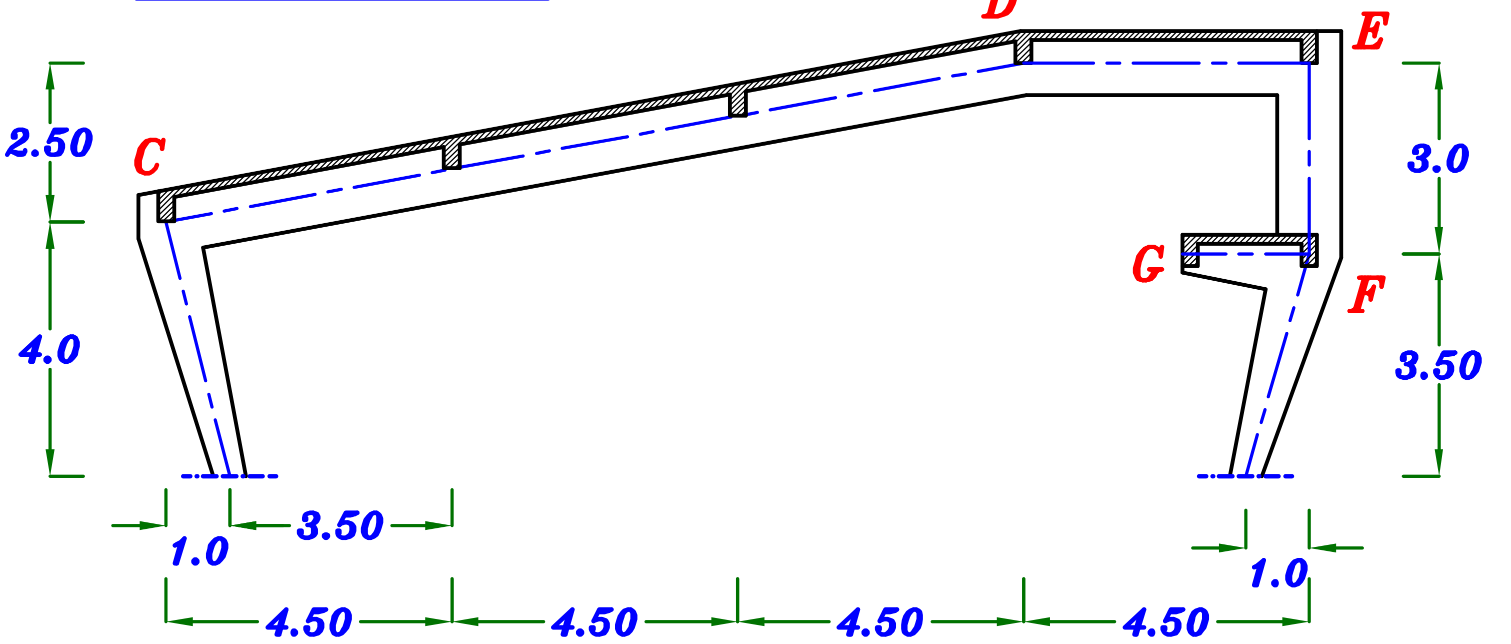


Sec. (4–4)



Sec. (5–5)

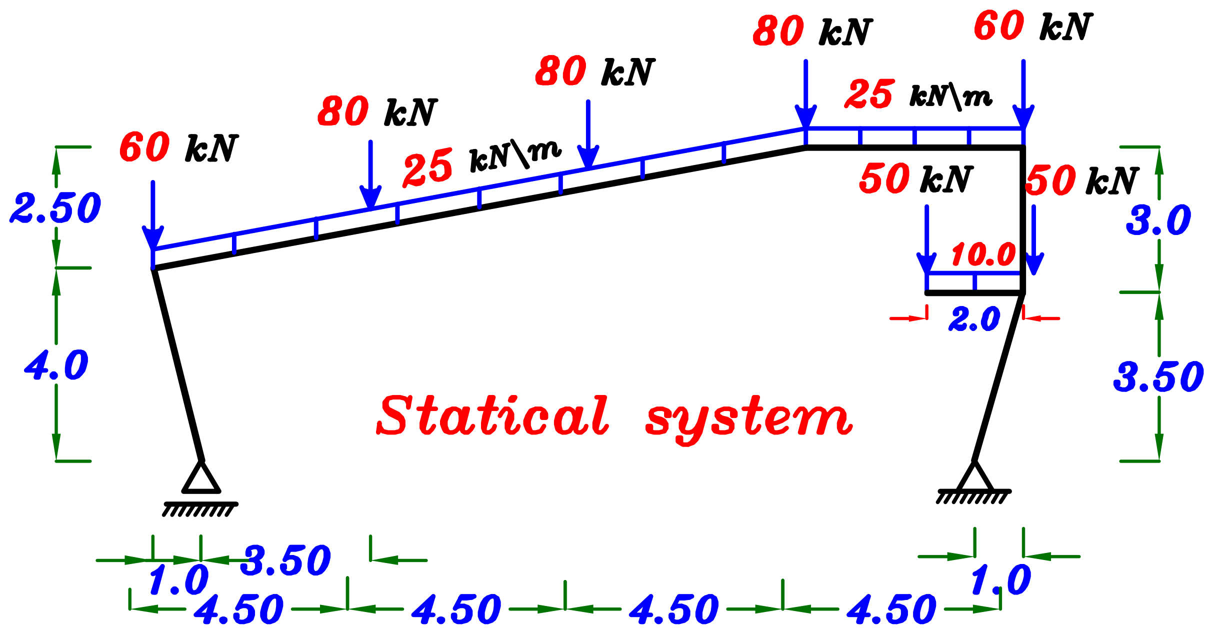
Example.



Data.

$F_{cu} = 30 \ N\backslash mm^2$ $\quad F_y = 360 \ N\backslash mm^2$

$t_s = 120 \ mm$ $\quad$ Spacing $= 6.0 \ m$



Req.

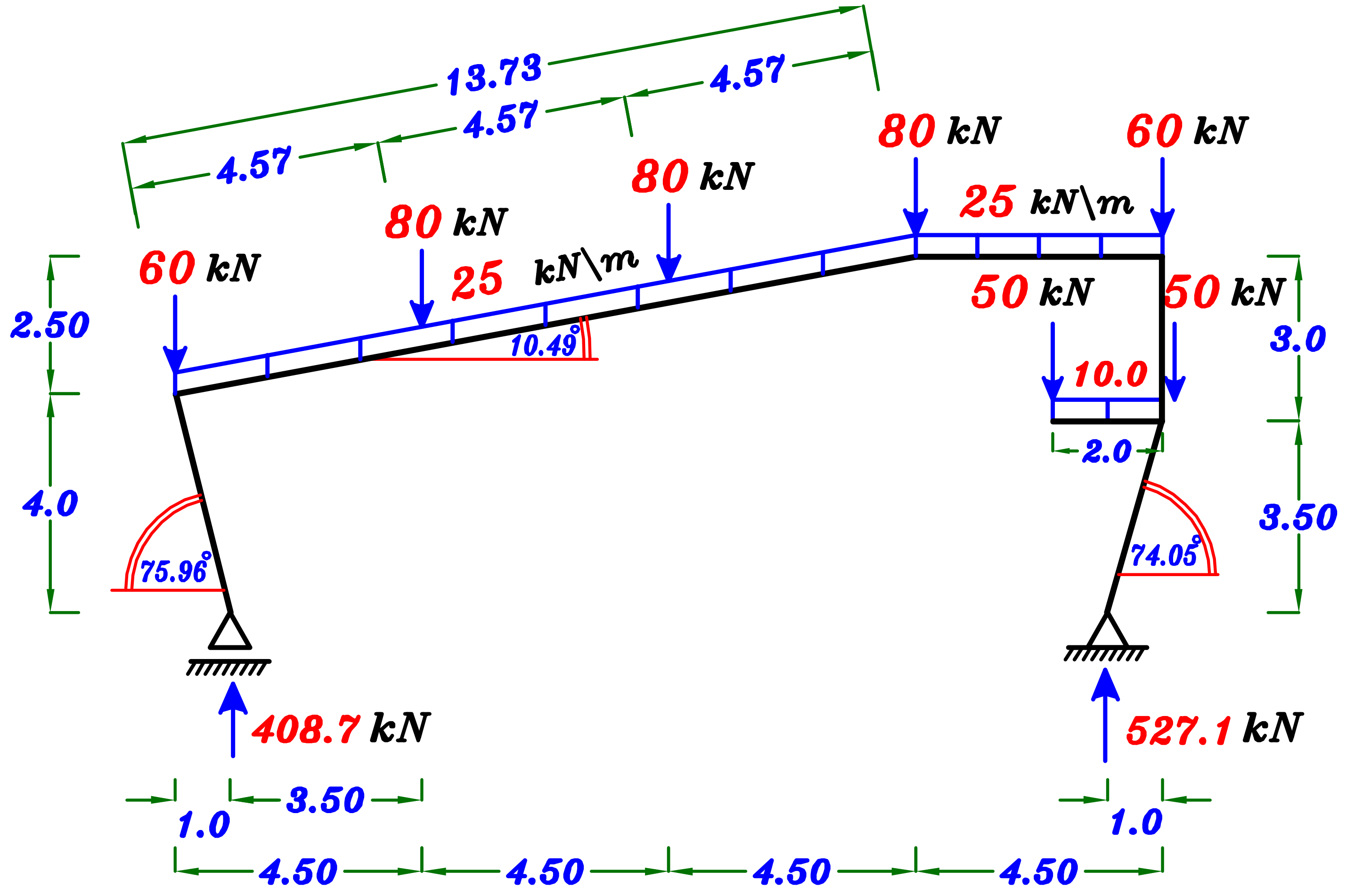
1 - For the given working loads, draw the bending moment Shear Force, Normal Force diagrams of the Frame.

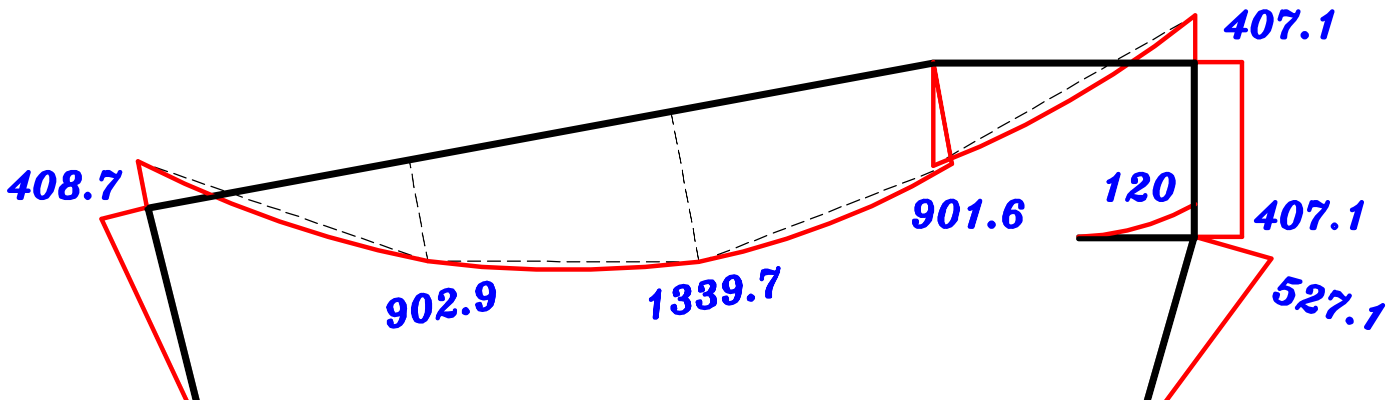
2 - Using Ultimate Limit Design Method, design the critical sections of the Frame.

3 - Check shear stresses For part (F G) of the Frame and Find the required shear reinforcement if necessary.

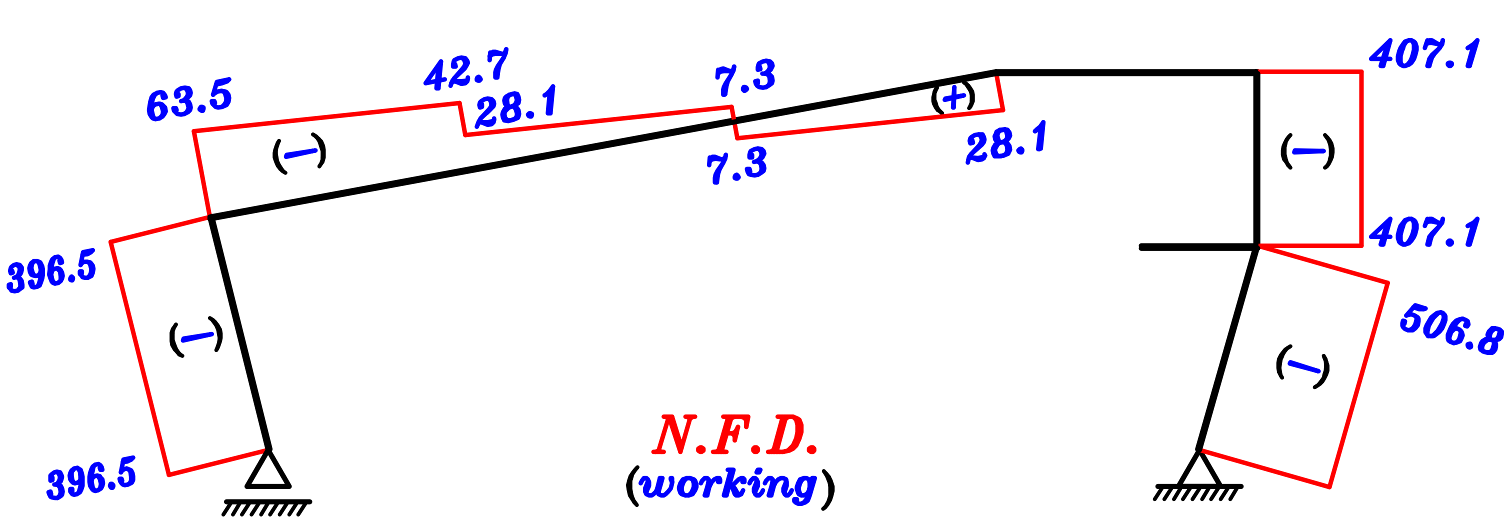
4 - Draw moment of resistance diagram For part (C D E) only.

5 - Draw to scale 1:50 an elevation of the Frame showing clearly the arrangement of the reinforcement. Draw to scale 1:25 cross-sections For the designed critical sections.

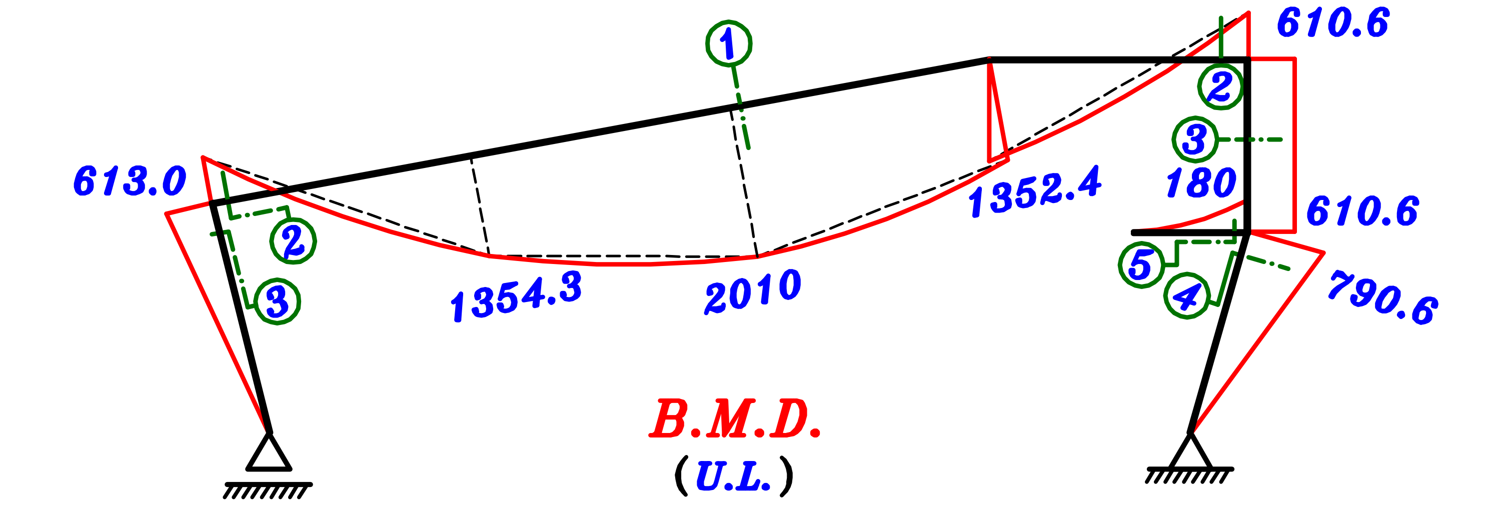


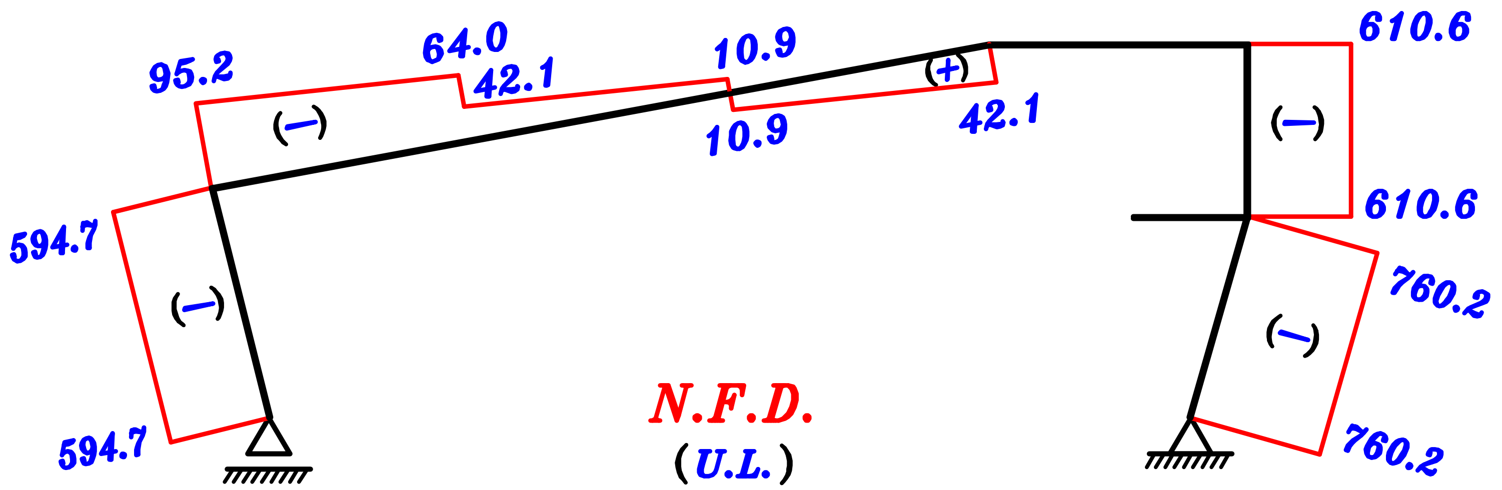


B.M.D.
(working)



N.F.D.
(working)


613.0
1354.3
2010
1352.4
610.6
180
610.6
790.6
1
2
3
2
3
5
4
B.M.D.
(U.L.)

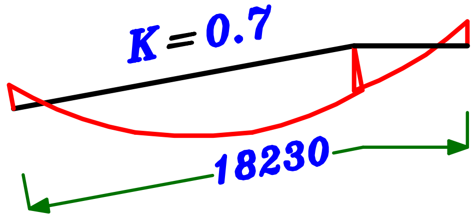

95.2
64.0
42.1
10.9
10.9
42.1
(+)
(–)
610.6
(–)
610.6
594.7
(–)
594.7
760.2
(–)
760.2
N.F.D.
(U.L.)

Design of Sections.

$\boxed{\text{Take } b_{(Frame)} = 400 \text{ mm}}$

Sec. ① $M = 2010$ kN.m , $T = 10.9$ kN (Can be neglected)

The sec. will be T–sec.



$$B = \begin{cases} C.L. - C.L. = \text{spacing} = 6.0\,m = 6000\,m \\ 16\,t_s + b = 16 * 120 + 400 = 2320 \text{ mm} \\ K\,\frac{L}{5} + b = 0.7 * \frac{18230}{5} + 400 = 2952 \text{ mm} \end{cases}$$

$\boxed{B = 2320 \text{ mm}}$

Take $C_1 = 6.0 \longrightarrow J = 0.826$

$$\therefore d = 6.0\sqrt{\frac{2010 * 10^6}{30 * 2320}} = 1019 \simeq 1000 \text{ mm}$$

$\therefore$ Take $\boxed{d = 1000 \text{ mm}}$, $\boxed{t = 1100 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{2010 * 10^6}{0.826 * 360 * 1000} = 6759 \text{ mm}^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 6759 \text{ mm}^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$ $\therefore$ Take $A_s = A_{s_{req.}} = 6759 \text{ mm}^2$ $14\phi 25$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\,A_s = (0.1 \rightarrow 0.2)\ 6759$ $6\phi 12$

Sec. ② $M = 613.0$ kN.m , $P = 95.2$ kN , $b = 400$ mm

$d = 1000$ mm (the same depth of Sec. ①)

Check $\frac{P}{F_{cu}\, b\, t} = \frac{95.2 * 10^3}{30 * 400 * 1100} = 0.0072 < 0.04$ $\therefore$ (neglect P)

$\therefore d = c_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}}$ $\therefore 1000 = C_1 \sqrt{\frac{613.0 * 10^6}{30 * 400}} \rightarrow C_1 = 4.42 \rightarrow J = 0.816$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{613.0 * 10^6}{0.816 * 360 * 1000} = 2086 \ mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 2086 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2086 \ mm^2$ 5ϕ25

Sec. ③ R-Sec. $M = 613.0$ kN.m , $P = 594.7$ kN

$d_o = 3.5 \sqrt{\frac{613.0 * 10^6}{30 * 400}} = 791$ mm (as R-Sec.)

$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)(791) = (870 \rightarrow 1028)$ mm

$\therefore$ Take $\boxed{d = 1000 \ mm}$, $\boxed{t = 1100 \ mm}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{594.7 * 10^3}{30 * 400 * 1100} = 0.045 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$e = \frac{M}{P} = \frac{613.0}{594.7} = 1.03$ m $\therefore \frac{e}{t} = \frac{1.03}{1.10} = 0.93 > 0.5 \xrightarrow{Use} e_s$

$$e_s = e + \frac{t}{2} - c = 1.03 + \frac{1.10}{2} - 0.10 = 1.48 \ m$$

$$M_s = P * e_s = 594.7 * 1.48 = 880.1 \ kN.m$$

$$\therefore 1000 = C_1 \sqrt{\frac{880.1 * 10^6}{30 * 400}} \longrightarrow C_1 = 3.69 \longrightarrow J = 0.79$$

$$A_s = \frac{M_s}{J \ F_y \ d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{880.1 * 10^6}{0.79 * 360 * 1000} - \frac{594.7 * 10^3}{(360 \backslash 1.15)} = 1195 \ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1195 \ mm^2$

$$\mu_{min.} b \, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b \, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \ mm^2$$

$$\therefore \ \mu_{min.} b \, d > A_{s_{req.}} \xrightarrow{Use} A_{s_{min.}}$$

$$A_{s_{min.}} = \left. \begin{array}{l} \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \ mm^2 \\ 1.3 \ A_{s_{req.}} = 1.3 * 1195 = 1553.5 \ mm^2 \end{array} \right\} \text{الأقل} = 1369.3 \ mm^2$$

$3 \phi 25$

Sec. 4 $\qquad$ R-Sec. $\quad M = 790.6 \ kN.m \ , \ P = 760.2 \ kN$

$d = 1000 \ mm$ (the same depth of Sec. 3)

Check $\frac{P}{F_{cu} \, b \, t} = \frac{760.2 * 10^3}{30 * 400 * 1100} = 0.057 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{790.6}{760.2} = 1.04 \ m \qquad \therefore \ \frac{e}{t} = \frac{1.04}{1.10} = 0.94 > 0.5 \xrightarrow{Use} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.04 + \frac{1.10}{2} - 0.10 = 1.49 \ m$$

$$M_s = P * e_s = 760.2 * 1.49 = 1132.7 \text{ kN.m}$$

$$\therefore\ 1000 = C_1 \sqrt{\frac{1132.7 * 10^6}{30 * 400}} \longrightarrow C_1 = 3.25 \longrightarrow J = 0.765$$

$$A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)} = \frac{1132.7 * 10^6}{0.765 * 360 * 1000} - \frac{760.2 * 10^3}{(360 \backslash 1.15)} = 1685 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1685 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1685 \text{ mm}^2$ $\quad$ $4\phi25$

Sec. ⑤ $\qquad M = 180 \text{ kN.m}, \quad b = 400 \text{ mm}$

$$d = 3.5 \sqrt{\frac{180 * 10^6}{30 * 400}} = 428.6 \text{ mm} \quad (\text{as R-Sec.})$$

$\therefore$ Take $\boxed{d = 450 \text{ mm}}$, $\boxed{t = 500 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{180 * 10^6}{0.78 * 360 * 428.6} = 1495 \text{ mm}^2$$

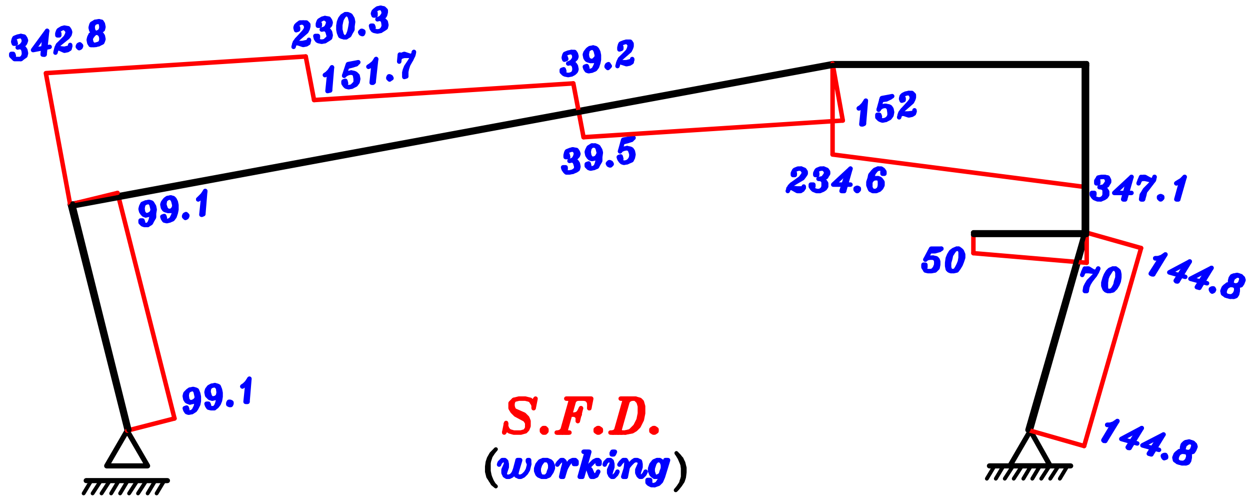
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1495 \text{ mm}^2$

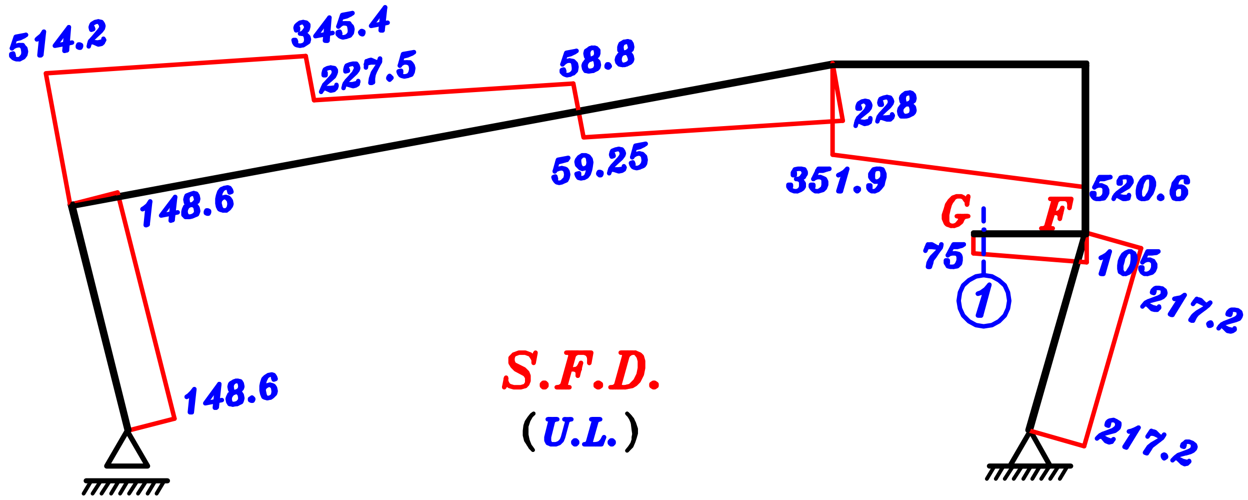
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1000 = 1369.3 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1495 \text{ mm}^2$ $\quad$ $4\phi25$

Take $Y = 0.40$ m

Check Shear. For part ((F G))





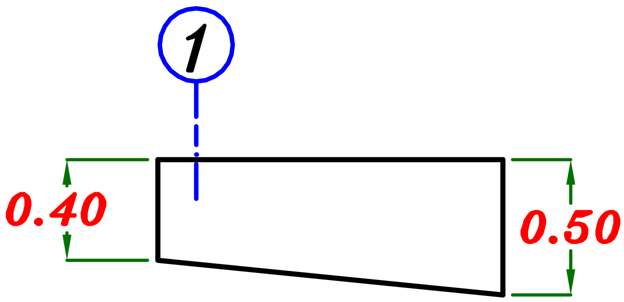
- Allowable shear stress.

- $q_{cu} = 0.24 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\dfrac{30}{1.5}} = 1.07 \quad N\backslash mm^2$

- $q_{max.} = 0.70 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.70 \sqrt{\dfrac{30}{1.5}} = 3.13 \quad N\backslash mm^2$

Sec. 1 $Q = 75.0$ kN

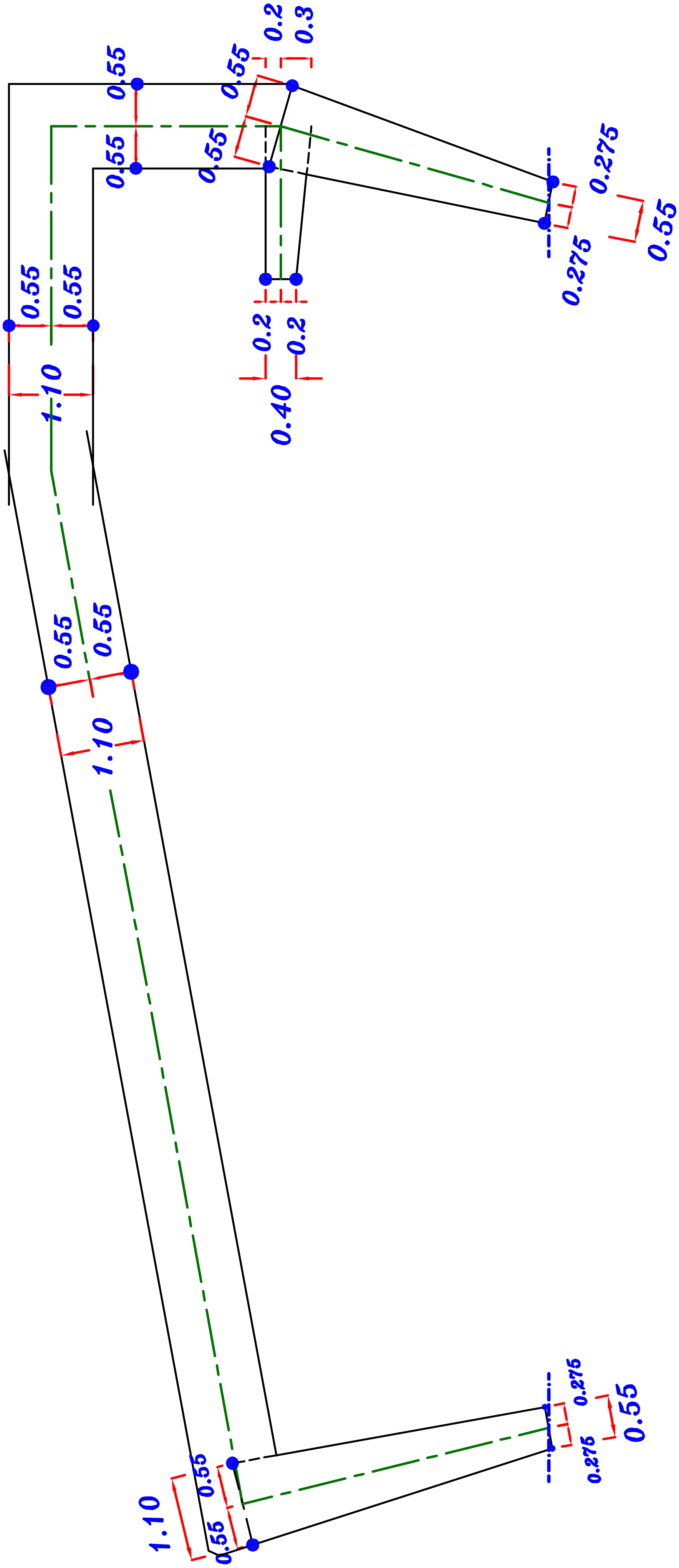
$\therefore$ Actual shear stress. =



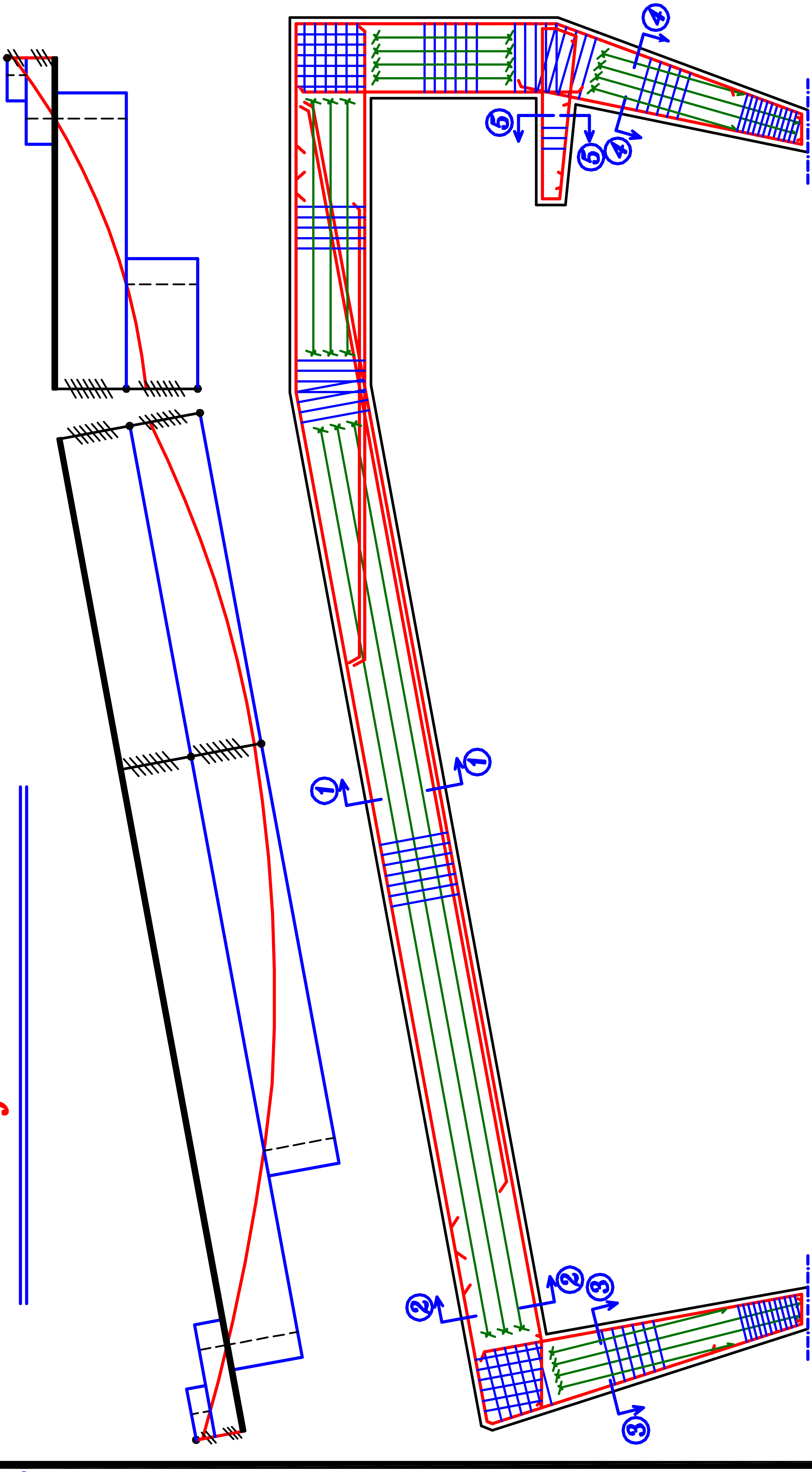
$$q_U = \frac{Q}{b\,d} - \frac{M \tan \beta}{b\,d^2}$$

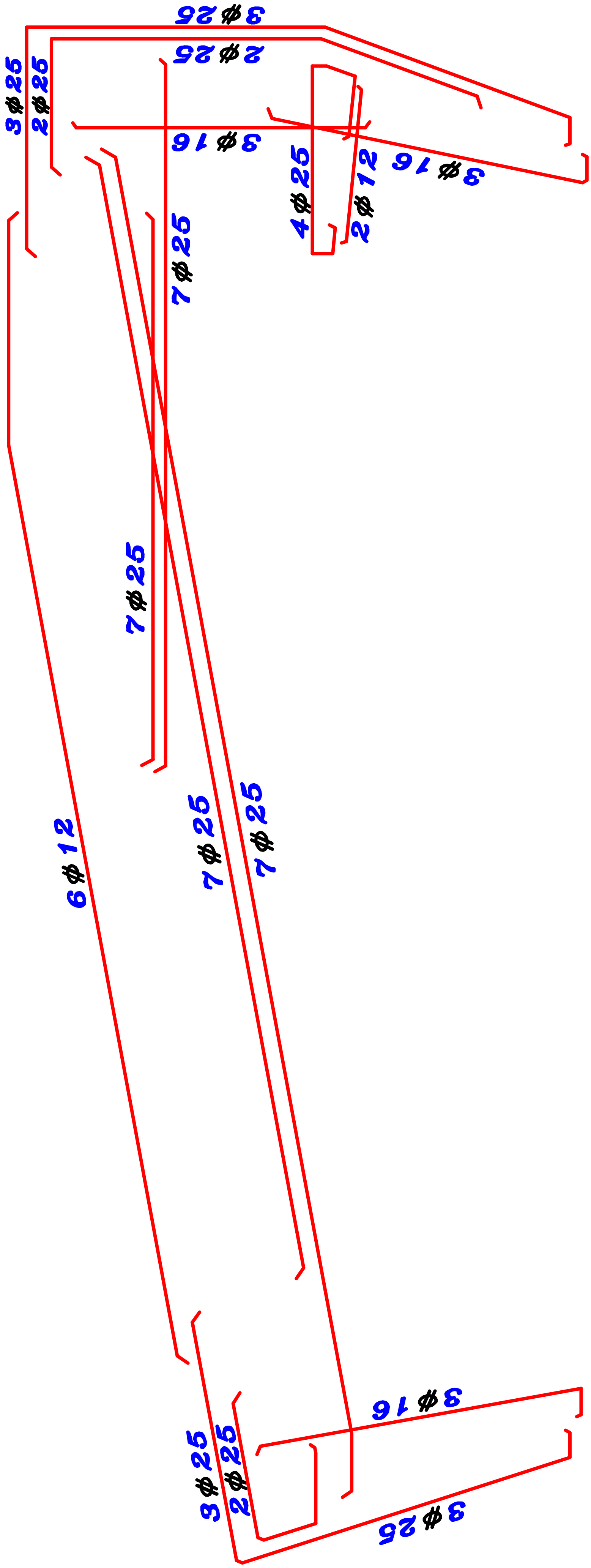
$$= \frac{75.0 * 10^3}{400 * 350} - ZERO = 0.535 \; N\backslash mm^2$$

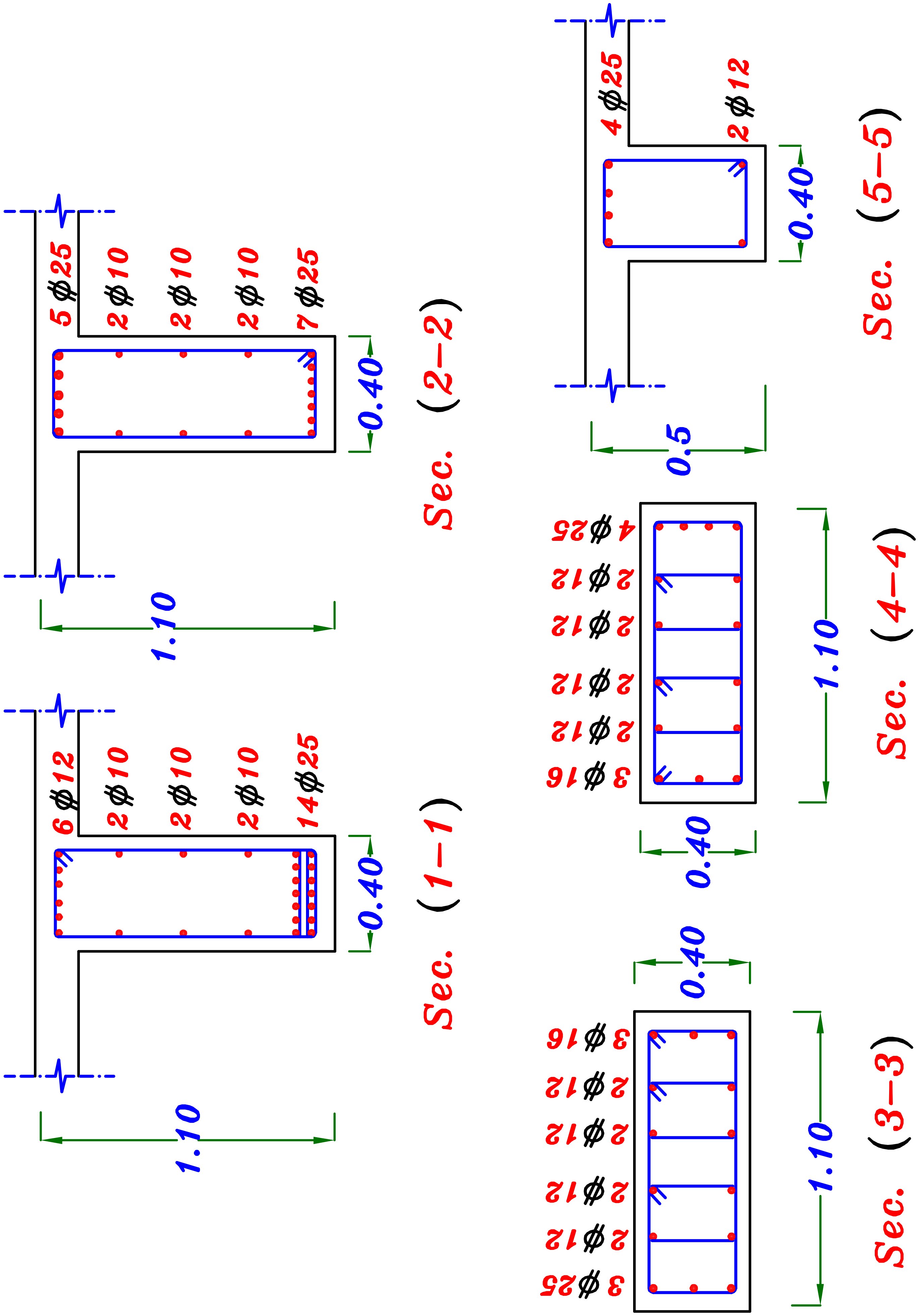
$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m'}$


0.55
0.55
0.55
0.55
0.2
0.3
0.55
0.55
0.275
0.275
0.55
0.2
0.2
0.40
1.10
0.55
0.55
1.10
0.275
0.275
0.55
0.55
0.55
1.10

R.F.T. of the Frame.




3 ϕ 25
2 ϕ 25
3 ϕ 25
2 ϕ 25
3 ϕ 16
3 ϕ 16
4 ϕ 25
2 ϕ 12
7 ϕ 25
7 ϕ 25
6 ϕ 12
7 ϕ 25
7 ϕ 25
3 ϕ 16
3 ϕ 25
2 ϕ 25
3 ϕ 25



Sec. (1-1)
6 ϕ12
2 ϕ10
2 ϕ10
2 ϕ10
14ϕ25
0.40
1.10
Sec. (2-2)
5 ϕ25
2 ϕ10
2 ϕ10
2 ϕ10
7 ϕ25
0.40
1.10
Sec. (3-3)
3 ϕ16
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
3 ϕ25
0.40
1.10
Sec. (4-4)
4 ϕ25
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
3 ϕ16
0.40
1.10
Sec. (5-5)
4 ϕ25
2 ϕ12
0.40
0.5

Example.

Figure (1) shows a sectional elevation of a reinforced concrete structure, The structure consists of reinforced concrete slabs supported by a system of secondary beams and Frame (F), spaced at 5.0 m It is required to :

1- Calculate the equivalent working loads For shear and moment For Beam (B).
2- Using the limit state design method (L.S.D.M.) , design the critical sections For the Beam (B) to satisfy the internal Forces requirements and then draw its details of reinforcement in elevation to scale 1:50 and cross sections to scale 1:25 Imperical curtailment of bars is required.
3- Draw the N.F.D. , S.F.D. & B.M.D. For the intermediate Frame (F) . Using the given working loads.
4- Design the critical sections of Frame (F) to satisfy the internal Forces requirments using (L.S.D.M.)
5- Draw details of reinforcement For Frame (F) in elevation to scale 1:50 and cross sections to scale 1:25 Curtailment of bars using the moment of resistance diagram is required.

Data:

- Concrete characteristic strength F_{cu} = 25 N/mm²
- Slab thickness t_s = 150 mm
- Live Load on slabs L.L. = 5 kN/m²/HL. projection
- Own weight of Frames = 7.0 kN/m`
- Steel used is St. 360/520
- Floor cover F.C. = 2.0 kN/m²
- Own weight of beams = 3.5 kN/m`
- γ_{brick} = 16.0 kN/m³

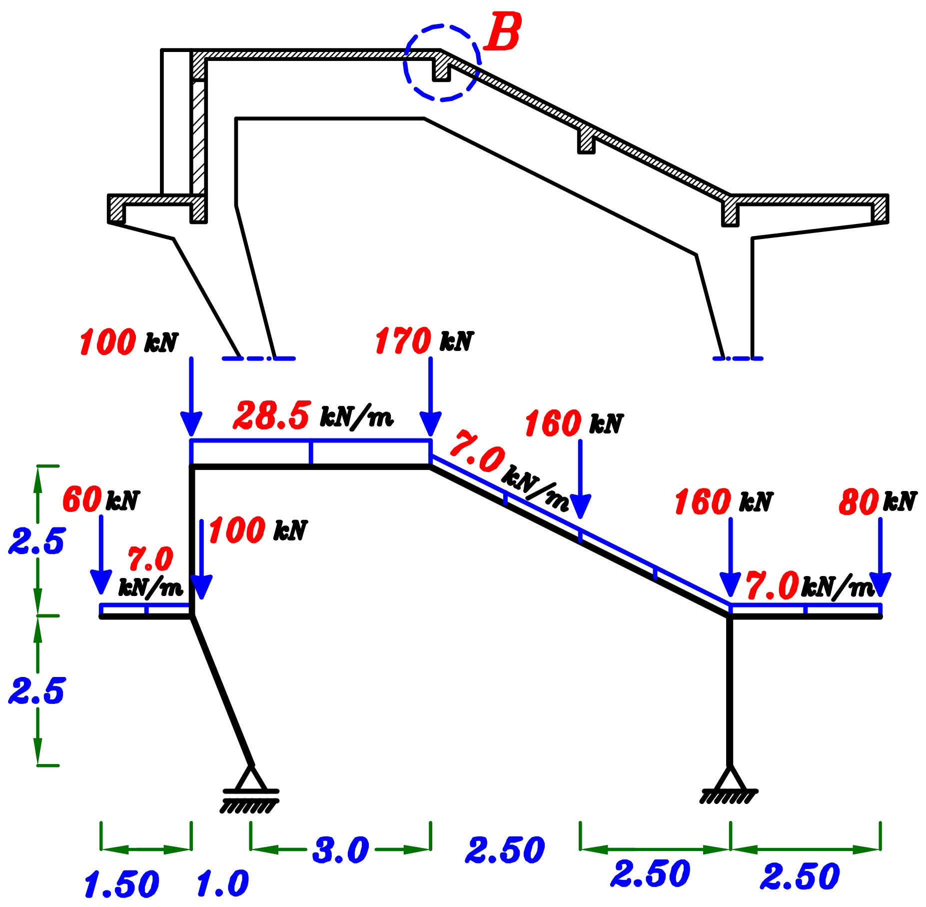
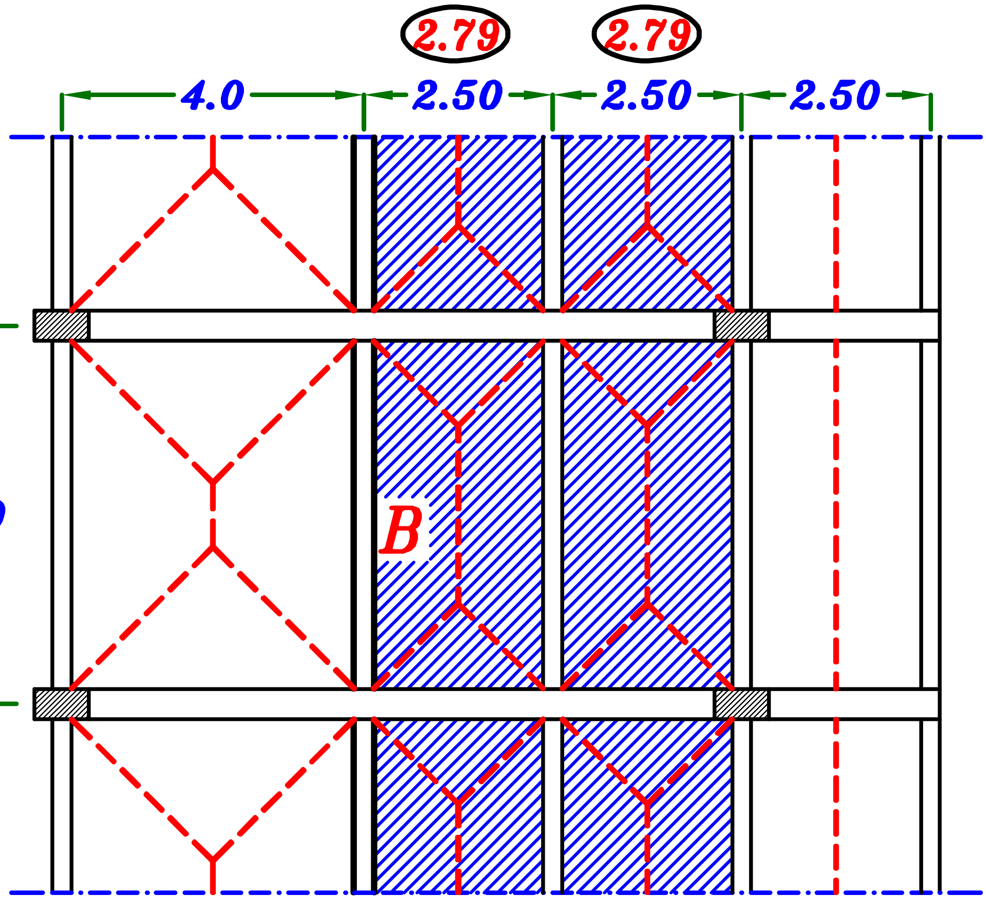


Figure (1)

1- Calculate the equivalent working loads For shear and moment For Beam (B).

$$\theta = 26.56^{\circ}$$



* **For Horizontal Slab.**

$$w_{sh} = o.w. + F.C. + L.L. = 0.15 * 25 + 2.0 + 5.0$$

$$\boxed{w_{sh} = 10.75 \ kN\backslash m^2}$$

$$C_a = 1 - \frac{1}{2}\left(\frac{4.0}{5.0}\right) = 0.60 \qquad C_e = 1 - \frac{1}{3}\left(\frac{4.0}{5.0}\right)^2 = 0.78$$

* **For Inclined Slab.**

$$w_{si} = o.w. + F.C. + L.L. * \cos\theta$$

$$= 0.15 * 25 + 2.0 + 5.0 * \cos 26.56^{\circ} = 10.22 \ kN\backslash m^2$$

$$\boxed{w_{si} = 10.22 \ kN\backslash m^2}$$

$$C_a = 1 - \frac{1}{2}\left(\frac{2.79}{5.0}\right) = 0.72 \qquad C_e = 1 - \frac{1}{3}\left(\frac{2.79}{5.0}\right)^2 = 0.89$$

$$w_a = O.W. + C_a\, w_{sh}\,\frac{L_s}{2} + C_a\, w_{si}\,\frac{L_s}{2}$$

$$= 3.0 + (0.60)(10.75)\left(\frac{4.0}{2}\right) + (0.72)(10.22)\left(\frac{2.79}{2}\right)$$

$$= 26.16 \ \ kN\backslash m`$$

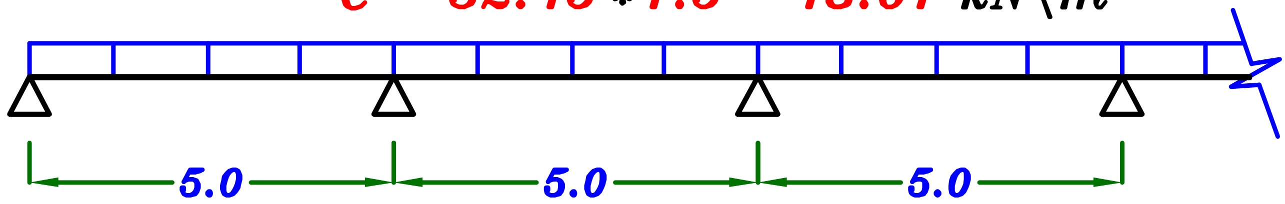
$$w_e = O.W. + C_e\, w_{sh}\,\frac{L_s}{2} + C_e\, w_{si}\,\frac{L_s}{2}$$

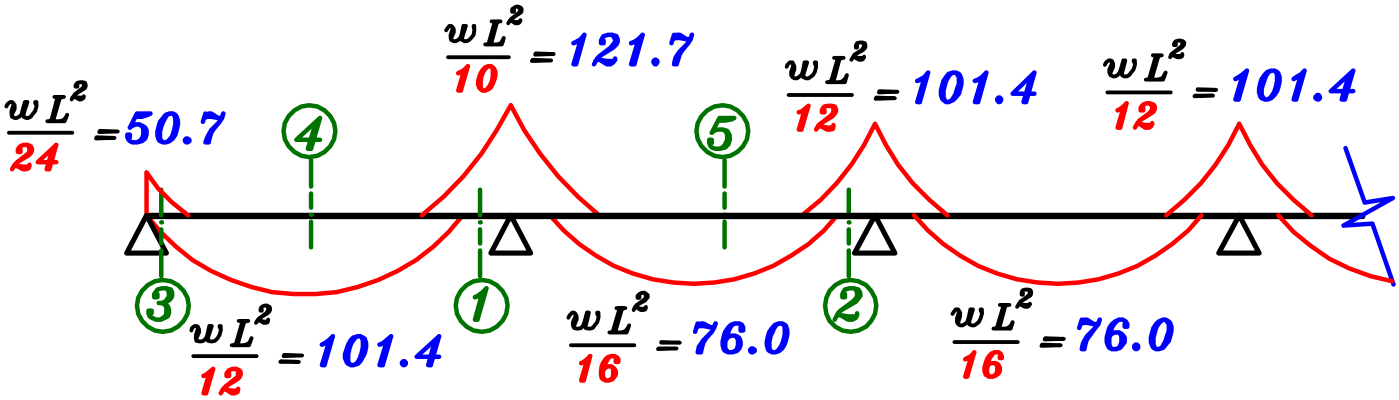
$$= 3.0 + (0.78)(10.75)\left(\frac{4.0}{2}\right) + (0.89)(10.22)\left(\frac{2.79}{2}\right)$$

$$= 32.45 \ \ kN\backslash m`$$

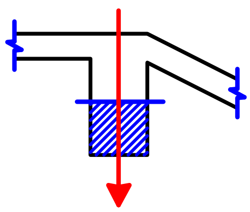
2- Using the limit state design method (L.S.D.M.), design the critical sections For the Beam (B) to satisfy the internal Forces requirements and then draw its details of reinforcement in elevation to scale 1:50 and cross sections to scale 1:25 Imperical curtailment of bars is required.

$$w_e = 32.45 * 1.5 = 48.67 \ kN\backslash m`$$

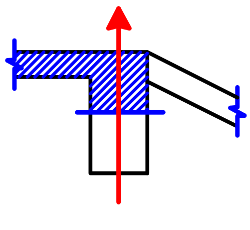




Sec. 1 $\quad M_{U.L.} = 121.7 \ \ kN.m \quad$ **R-Sec.**



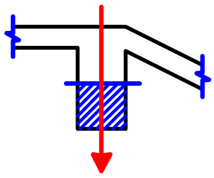
Sec. 4 $\quad M_{U.L.} = 101.4 \ \ kN.m \quad$ **L-Sec.**



$\because M_L < 2\,M_R \quad \therefore$ **Design R-Sec. First.**

Sec. ①

$M_{U.L.} = 121.7 \; kN.m$ R-Sec.



- Take C_1 between $(3.0 \rightarrow 4.0)$ $C_1 = 3.50$
- From charts $C_1 = 3.50 \longrightarrow J = 0.78$
- Get $d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\frac{121.7 * 10^6}{25 * 250}} = 488.4 \; mm$
- Take $\boxed{d = 500 \; mm}$, $\boxed{t = 550 \; mm}$

$$A_s = \frac{M_{U.L.}}{J \, F_y \, d} = \frac{121.7 * 10^6}{0.78 * 360 * 488.4} = 887.4 \; mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 887.4 \; mm^2$

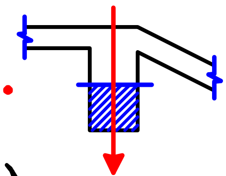
$$\mu_{min.} \, b \, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b \, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6 \; mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.} b \, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 887.4 \; mm^2$ 5 ϕ16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \text{ bars}$$

Sec. ②

$M_{U.L.} = 101.4 \; kN.m$ R-Sec.



Take $d = 0.50 \; m$ (The same d of Sec. ①)

$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}}$ $\therefore 350 = C_1 \sqrt{\frac{101.4 * 10^6}{25 * 250}} \longrightarrow C_1 = 3.92 \longrightarrow J = 0.801$

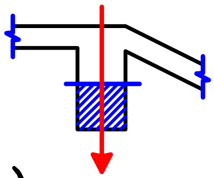
$$\therefore A_s = \frac{M_{U.L.}}{J \, F_y \, d} = \frac{101.4 * 10^6}{0.801 * 360 * 500} = 703.3 \; mm^2$$

<u>Check $A_{s_{min.}}$</u> $\qquad A_{s_{req.}} = 703.3 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6 \ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 703.3 \ mm^2 \qquad 4 \phi 16$$

<u>Sec. ③</u> $\qquad M_{U.L.} = 50.7 \ kN.m \qquad$ R-Sec.



Take $d = 0.50 \ m$ (The same d of Sec. ①)

$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 350 = C_1 \sqrt{\frac{50.7 * 10^6}{25 * 250}} \longrightarrow C_1 = 5.55 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{50.7 * 10^6}{0.826 * 360 * 500} = 341.0 \ mm^2$$

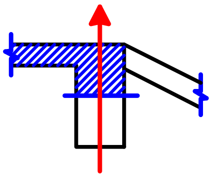
<u>Check $A_{s_{min.}}$</u> $\qquad A_{s_{req.}} = 341.0 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6 \ mm^2$$

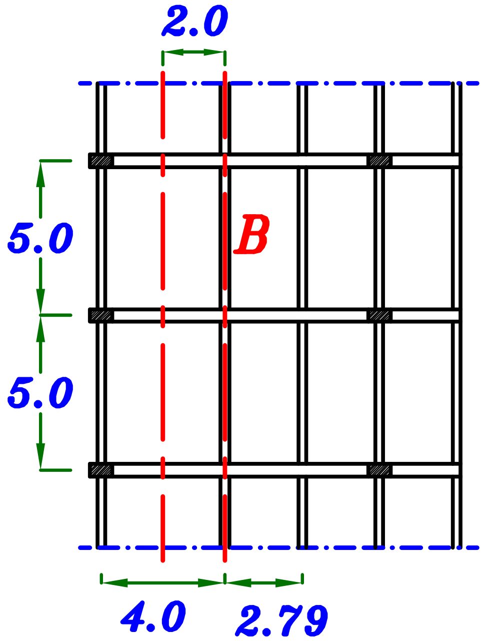
$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

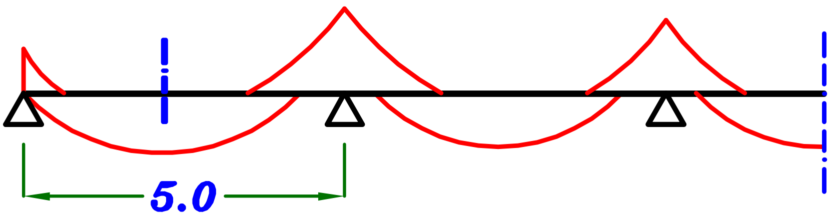
$$A_{s_{min.}} = \left.\begin{array}{l} \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6 \ mm^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 341.0 = 443.3 \ mm^2 \end{array}\right\} \text{الأقل} = 390.6 \ mm^2 \qquad 2 \phi 16$$

Sec. ④ $M_{U.L.} = 101.4$ kN.m L-Sec.



$\therefore$ Take $d = 0.50$ m (The same d of Sec. ①)





$$B = \begin{cases} C.L.-C.L. = 2.0\ m = 2000\ mm \\ 6\,t_s + b = 6 * 150 + 250 = 1150\ mm \\ K\dfrac{L}{10} + b = 0.8 * \dfrac{5000}{10} + 250 = 650\ mm \end{cases} \qquad \boxed{B = 650\ mm}$$

$$\because\ d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\,B}} \quad \therefore\ 500 = C_1\sqrt{\frac{101.4 * 10^6}{25 * 650}} \longrightarrow C_1 = 6.33 \longrightarrow J = 0.826$$

$$\therefore\ A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{101.4 * 10^6}{0.826 * 360 * 500} = 682\ \ mm^2$$

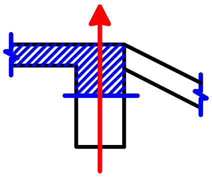
Check $A_{s_{min.}}$ $A_{s_{req.}} = 682\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6\ mm^2$$

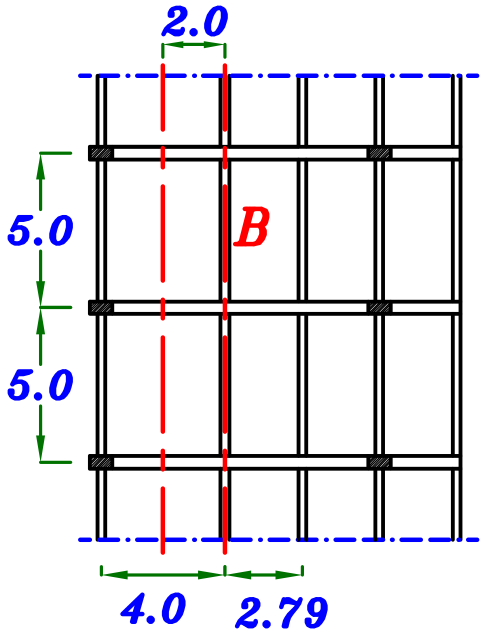
$$\therefore\ A_{s_{req.}} > \mu_{min.}\,b\,d \quad \therefore\ \text{Take } A_s = A_{s_{req.}} = 682\ mm^2 \qquad 4\phi16$$

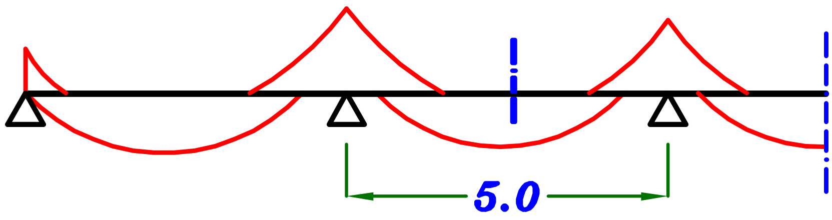
Sec. ⑤

$M_{U.L.} = 76.0$ kN.m   L−Sec.



Take $d = 0.50$ m (The same d of Sec. ①)





$$B = \begin{cases} C.L.-C.L. = 2.0\ m = 2000\ mm \\ 6\ t_s + b = 6 * 150 + 250 = 1150\ mm \\ K \frac{L}{10} + b = 0.7 * \frac{5000}{10} + 250 = 600\ mm \end{cases} \qquad \boxed{B = 600\ mm}$$

$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\ B}} \quad \therefore 500 = C_1 \sqrt{\frac{76.0 * 10^6}{25 * 600}} \longrightarrow C_1 = 7.02 \longrightarrow J = 0.826$$

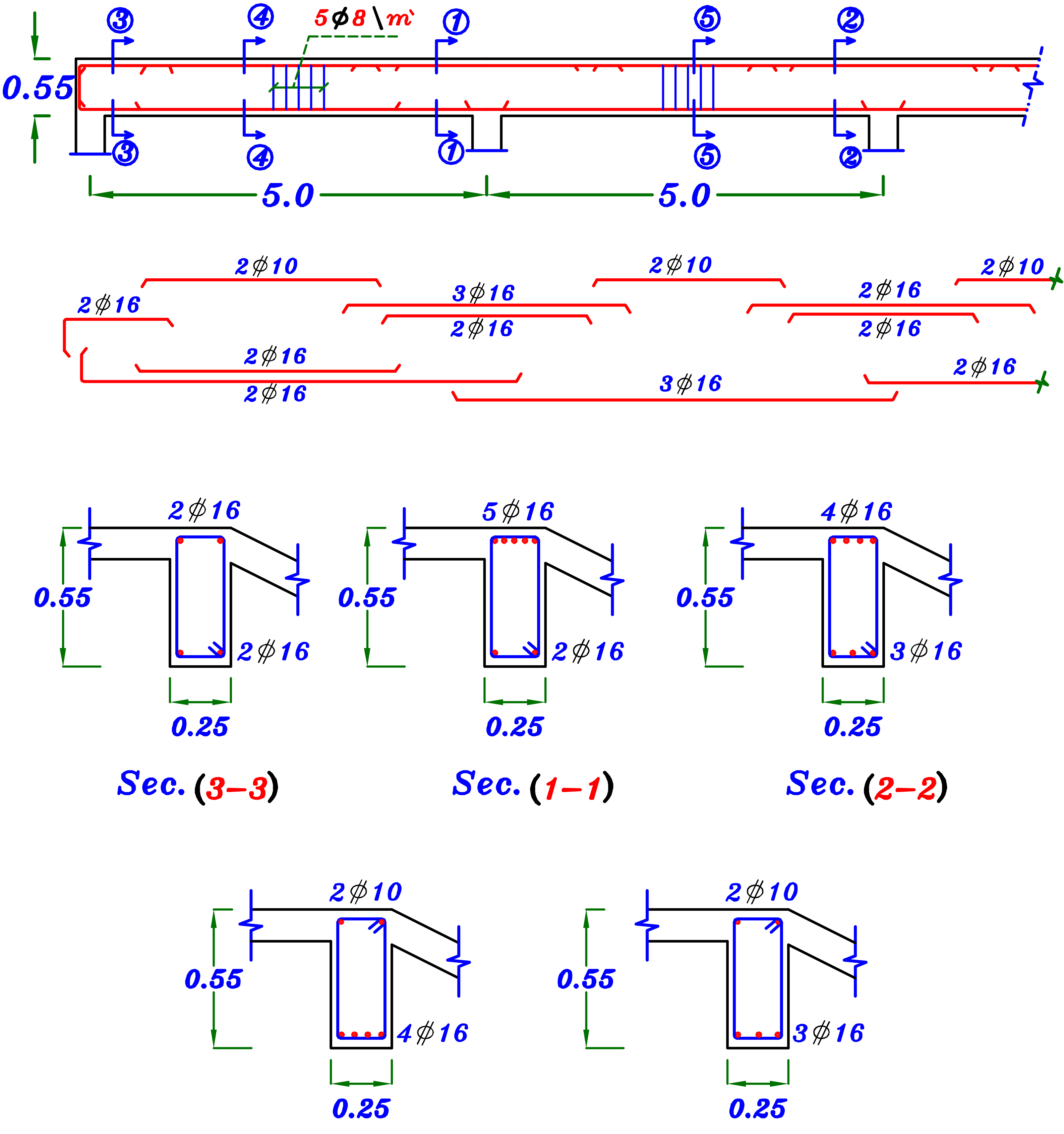
$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{76.0 * 10^6}{0.826 * 360 * 500} = 511.1\ mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 511.1\ mm^2$

$$\mu_{min.}\ b\ d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\ d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 500 = 390.6\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\ b\ d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 511.1\ mm^2 \quad 3\ \phi 16$$

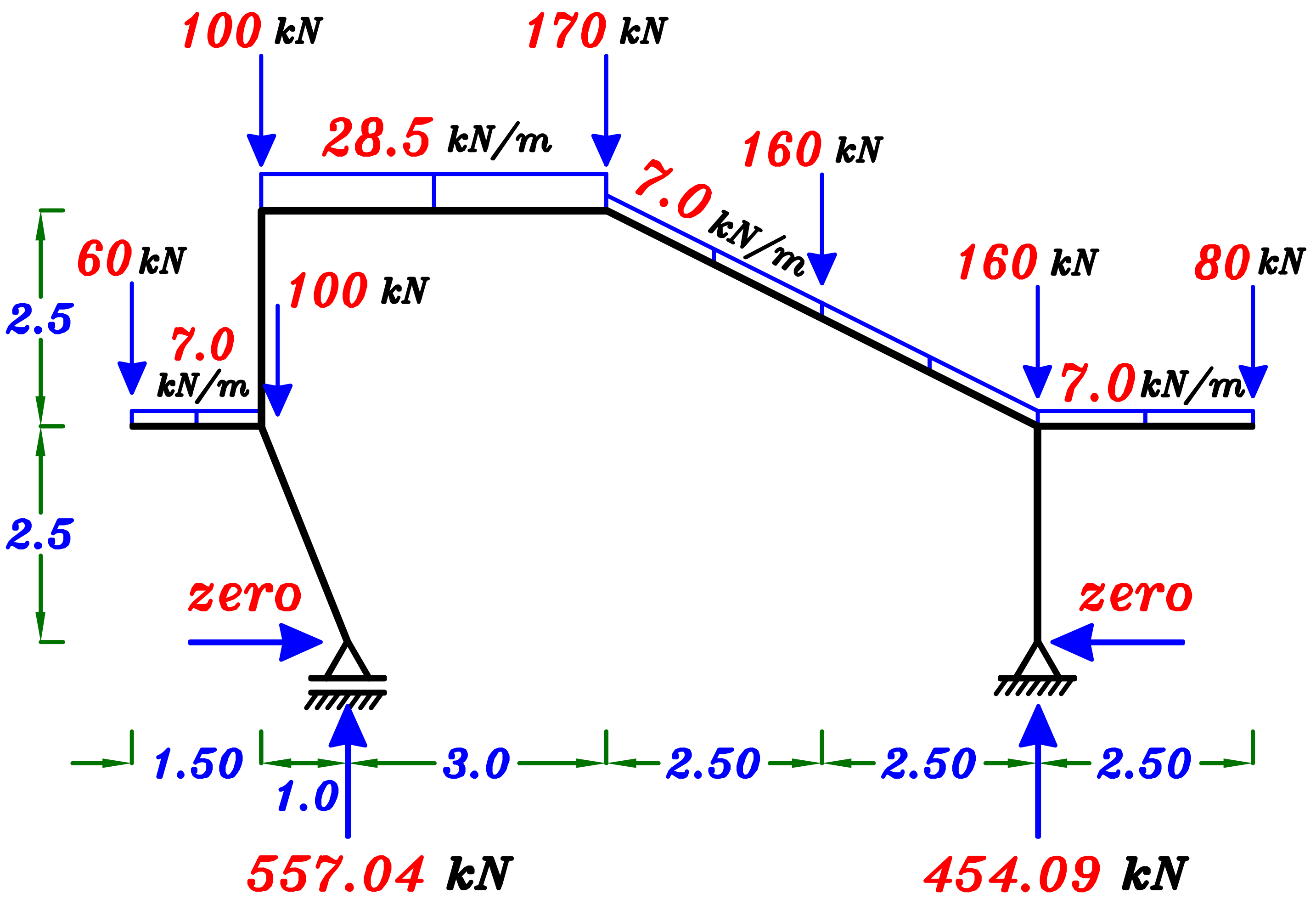
RFT. of Beam B

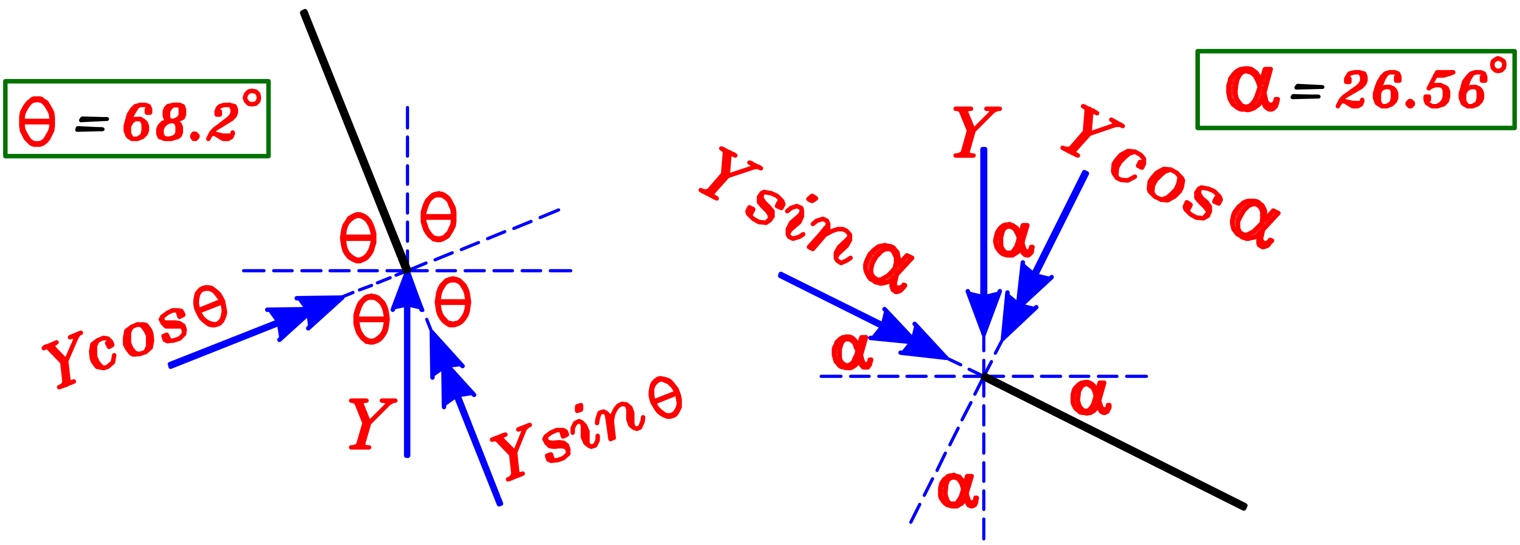


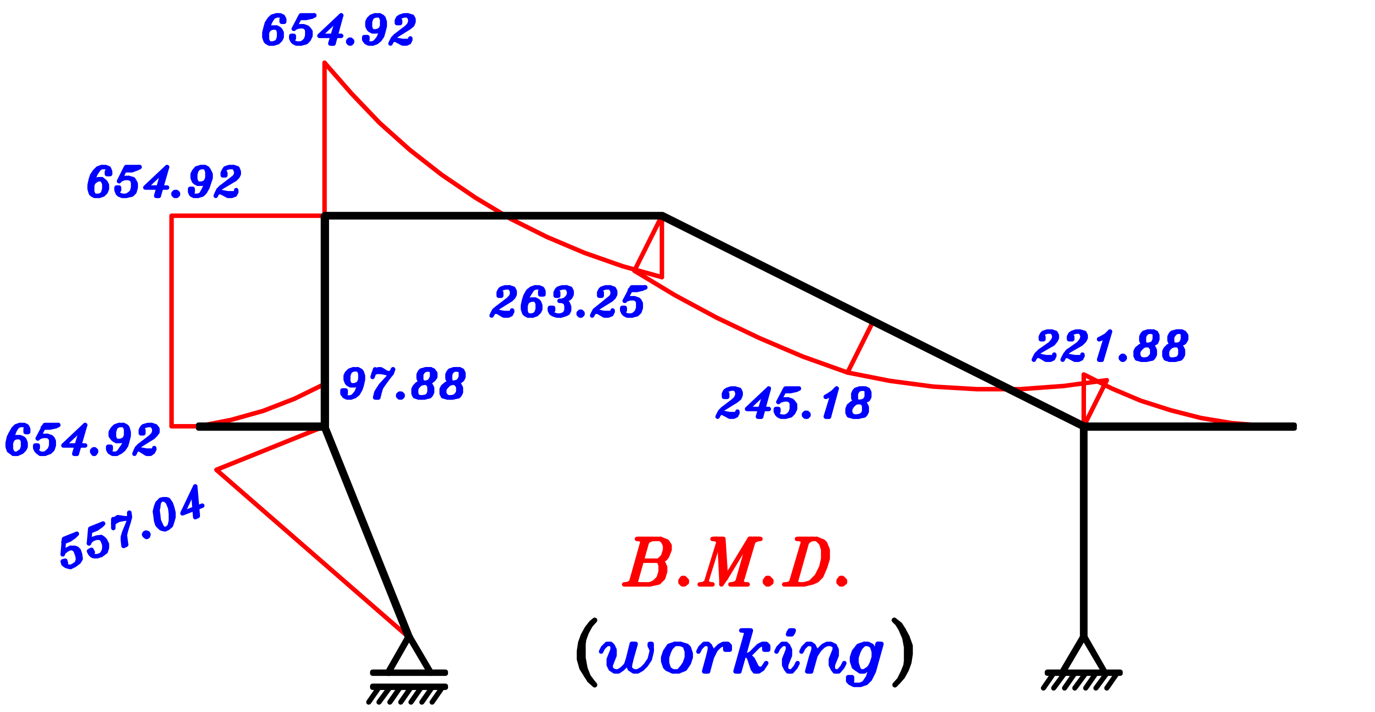
Sec. (3-3) Sec. (1-1) Sec. (2-2)

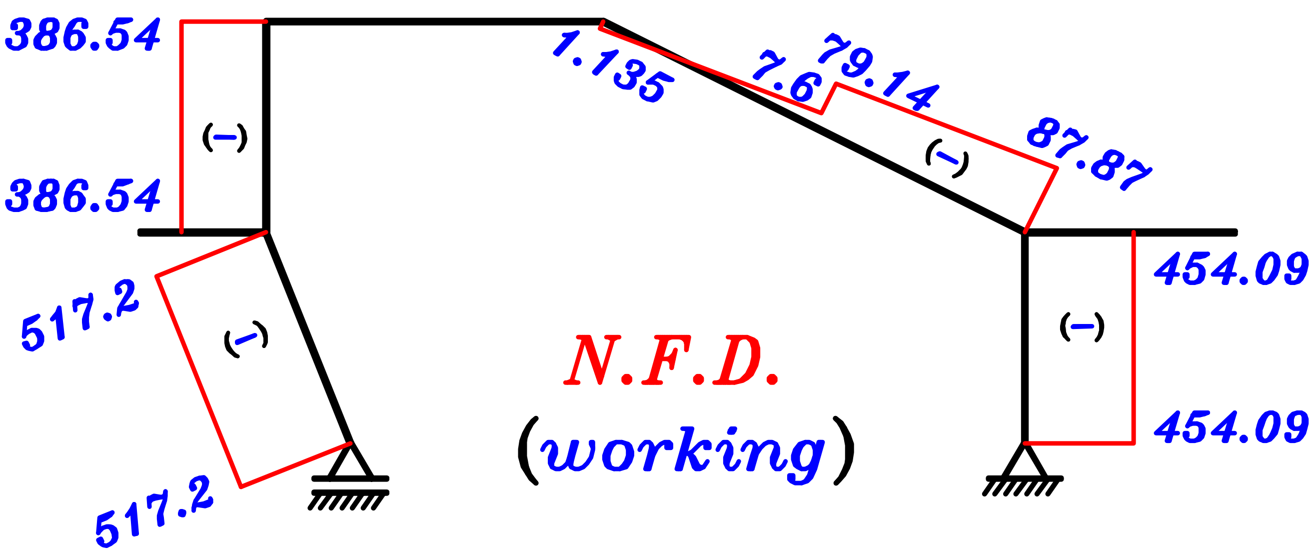
Sec. (4-4) Sec. (5-5)

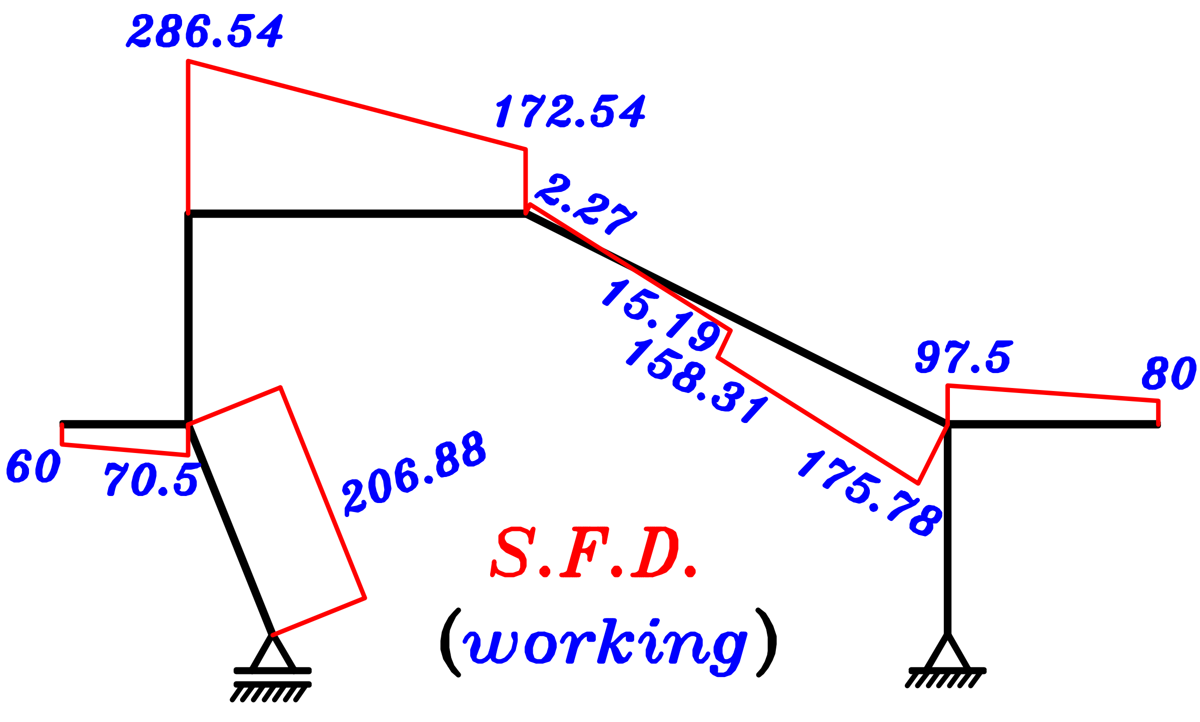
3- Draw the N.F.D. , S.F.D. & B.M.D. For the intermediate Frame (F).
Using the given working loads.

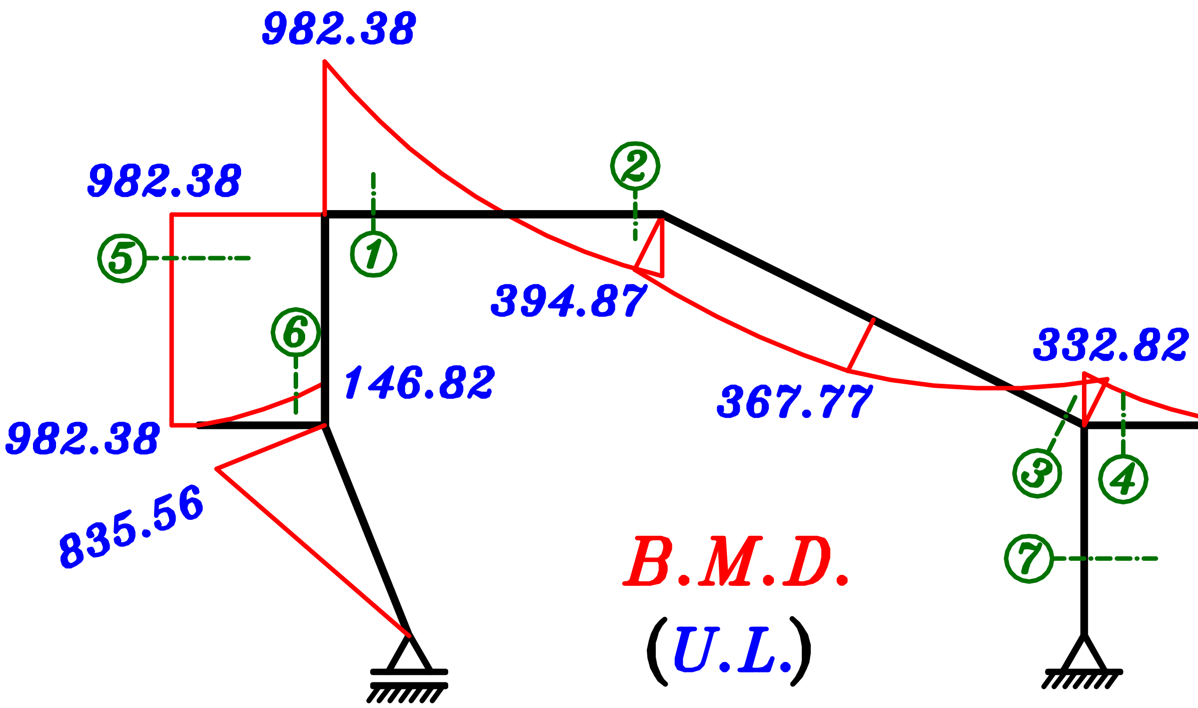


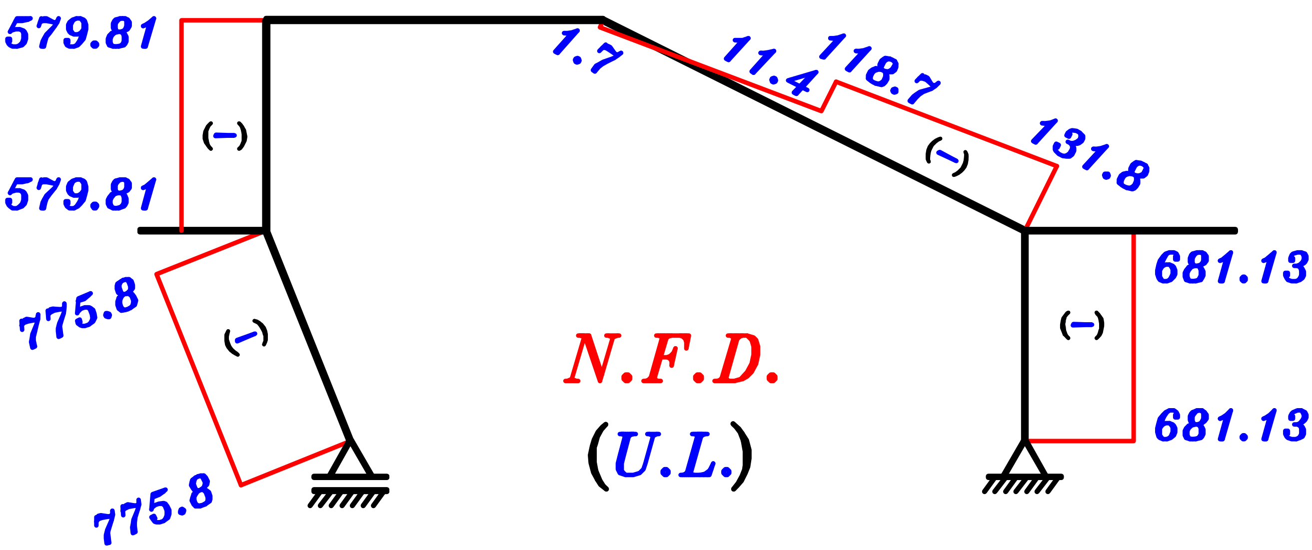


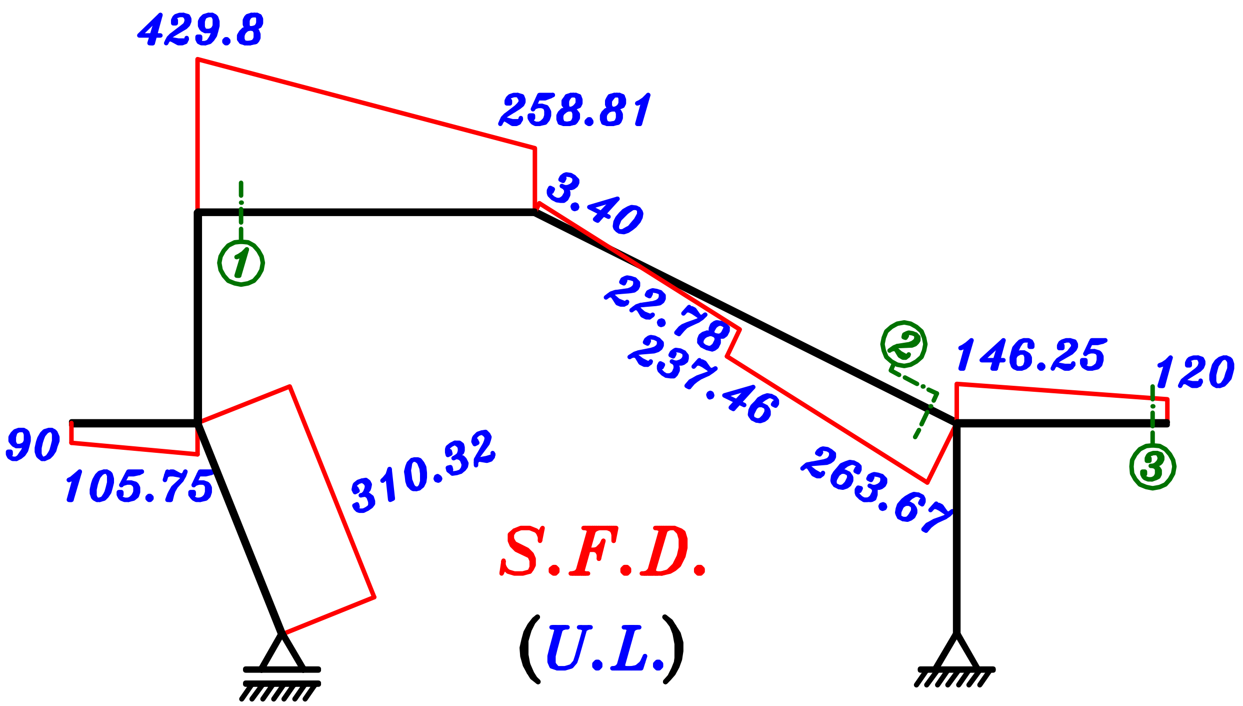

654.92
654.92
263.25
221.88
97.88
245.18
654.92
557.04
B.M.D.
(working)


386.54
1.135
7.6
79.14
(–)
87.87
386.54
454.09
517.2
(–)
(–)
N.F.D.
(working)
454.09
517.2


286.54
172.54
2.27
15.19
158.31
97.5
80
60
70.5
206.88
175.78
S.F.D.
(working)


982.38
982.38
394.87
146.82
367.77
332.82
982.38
835.56
1
2
3
4
5
6
7
B.M.D.
(U.L.)

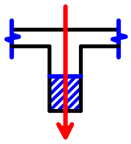

579.81
1.7
11.4
118.7
(–)
(–)
131.8
579.81
681.13
775.8
(–)
(–)
681.13
775.8
N.F.D.
(U.L.)


429.8
258.81
3.40
1
22.78
237.46
2
146.25
120
3
90
105.75
310.32
263.67
S.F.D.
(U.L.)

4- Design the critical sections of Frame (F) to satisfy the internal Forces requirements using (L.S.D.M.)

Take $b = 350$ mm

Sec. ① $M = 982.38$ kN.m , R-Sec.



Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$d = 3.5\sqrt{\frac{982.38*10^6}{25*350}} = 1172.7 \text{ mm}$$

Take $\boxed{d = 1200 \text{ mm}}$, $\boxed{t = 1300 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{982.38*10^6}{0.780*360*1172.7} = 2983.2 \text{ mm}^2$$

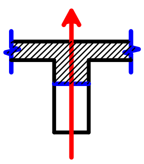
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 2983.2 \text{ mm}^2$

$$\mu_{min.}\,b\,d = \left(0.225*\frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225*\frac{\sqrt{25}}{360}\right) 350*1200 = 1312.5 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2983.2 \text{ mm}^2$ $\quad$ 8 ϕ 22

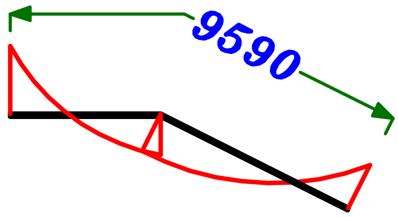
$$\therefore n = \frac{b-25}{\phi+25} = \frac{350-25}{22+25} = 6.91 = 6.0 \text{ bars}$$

Sec. ② $M = 394.87$ kN.m , T-Sec.



$d = 1200$ mm (the same depth of sec. ①)

$$B = \begin{cases} C.L.-C.L. = 5.0 \text{ m} = 5000 \text{ mm} \\ 16\,t_s + b = 16*150 + 350 = 2750 \text{ mm} \\ K\,\frac{L}{5} + b = 0.7*\frac{9590}{5} + 350 = 1692.6 \text{ mm} \end{cases}$$



$\boxed{B = 1692.6 \text{ mm}}$

$$\because\ d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}B}} \quad \therefore\ 1200 = C_1\sqrt{\frac{394.87*10^6}{25*1692.6}} \longrightarrow C_1 = 12.42 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{394.87*10^6}{0.826*360*1200} = 1106.6\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1106.6\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225*\frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225*\frac{\sqrt{25}}{360}\right) 350*1200 = 1312.5\ mm^2$$

$$\therefore\ \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{Use} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225*\frac{\sqrt{25}}{360}\right) 350*1200 = 1312.5\ mm^2 \quad \text{الأقل} \\ 1.3\,A_{s_{req.}} = 1.3*1106.6 = 1438.6\ mm^2 \end{array}\right\} = 363.3\ mm^2 \quad 4\ \phi\ 22$$

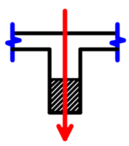
$$\text{Stirrup Hangers} = (0.1 \rightarrow 0.2)\ A_s = (0.1 \rightarrow 0.2)\ 1283.3 \quad 2\ \phi\ 12$$

Sec. ③ $\quad M = 332.82\ kN.m\ ,\ P = 131.8\ kN\ ,\ b = 350\ mm$

$d = 1200\ mm$ (the same depth of Sec. ①)

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{131.8*10^3}{25*350*1300} = 0.011 < 0.04 \quad (\text{Neglect } P)$$

The sec. will be R-sec.



$$\because\ d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore\ 1200 = C_1\sqrt{\frac{332.82*10^6}{25*350}} \longrightarrow C_1 = 6.15 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{332.82*10^6}{0.826*360*1200} = 932.7\ mm^2$$

Check $As_{min.}$ $\qquad As_{req.} = 932.7\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1200 = 1312.5\ mm^2$$

$$\therefore\ \mu_{min.}\, b\, d > As_{req.} \xrightarrow{Use} As_{min.}$$

$$\left.\begin{array}{l} As_{min.} = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1200 = 1312.5\ mm^2 \\ 1.3\, As_{req.} = 1.3 * 932.7 = 1212.5\ mm^2 \end{array}\right\} \text{الأقل} = 1212.5\ mm^2 \quad 4\phi22$$

Sec. ④ $\quad M = 332.82\ kN.m\ ,\ b = 350\ mm$

Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$d = 3.5\sqrt{\frac{332.82 * 10^6}{25 * 350}} = 682.6\ mm$$

$$\boxed{d = 700\ mm}\ ,\ \boxed{t = 750\ mm}$$

$$\therefore\ As = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{332.82 * 10^6}{0.780 * 360 * 682.6} = 1736.4\ mm^2$$

Check $As_{min.}$ $\qquad As_{req.} = 1736.4\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1200 = 1312.5\ mm^2$$

$$\therefore\ As_{req.} > \mu_{min.}\, b\, d \quad \therefore \text{Take } As = As_{req.} = 1736.4\ mm^2 \quad 5\phi22$$

$$Y = \left\{\begin{array}{l} \frac{t}{2} = \frac{750}{2} = 375\ mm \\ t_b = \frac{spacing}{12} = \frac{5000}{12} = 416.6\ mm \\ t - \frac{L_c}{3} = 750 - \frac{2500}{3} = -83.3\ mm \end{array}\right\} \quad \boxed{Y = 450\ mm}$$

تم تصميم القطاع Sec.④ غير Sec.③ لان له عمق مختلف .

Sec. ⑤ R−Sec. $M = 982.38$ kN.m , $P = 579.81$ kN

$$d_o = 3.5\sqrt{\frac{982.38 * 10^6}{25 * 350}} = 1172.7 \text{ mm} \quad (\text{as R−Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)(1172.7) = (1290 \rightarrow 1524) \text{ mm}$$

∴ Take $\boxed{d = 1300 \text{ mm}}$, $\boxed{t = 1400 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{579.81 * 10^3}{25 * 350 * 1400} = 0.047 > 0.04$ (Don't neglect P)

∴ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{982.38}{579.81} = 1.69 \text{ m} \quad ∴ \frac{e}{t} = \frac{1.69}{1.40} = 1.20 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.69 + \frac{1.40}{2} - 0.10 = 2.29 \text{ m}$$

$$M_s = P * e_s = 579.81 * 2.29 = 1327.76 \text{ kN.m}$$

$$∴ 1300 = C_1\sqrt{\frac{1327.76 * 10^6}{25 * 350}} \longrightarrow C_1 = 3.33 \longrightarrow J = 0.77$$

$$A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{1327.76 * 10^6}{0.77 * 360 * 1300} - \frac{579.81 * 10^3}{(360 \backslash 1.15)} = 1832.3 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1832.3 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1200 = 1312.5 \text{ mm}^2$$

$$∴ A_{s_{req.}} > \mu_{min.}\, b\, d \quad ∴ \text{Take } A_s = A_{s_{req.}} = 1832.3 \text{ mm}^2$$

5 ϕ 22

Sec. ⑥ $M = 146.82$ kN.m , $b = 350$ mm R-sec.

Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$d = 3.5 \sqrt{\frac{146.82 * 10^6}{25 * 350}} = 453.37 \text{ mm}$$

$\boxed{d = 500 \text{ mm}}$, $\boxed{t = 550 \text{ mm}}$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{146.82 * 10^6}{0.780 * 360 * 453.37} = 1153.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1153.2 \text{ mm}^2$

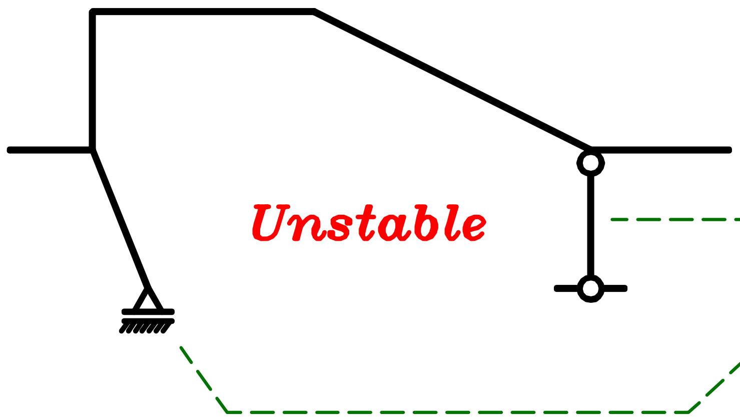
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1200 = 1312.5 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\quad \therefore$ Take $A_s = A_{s_{req.}} = 1153.2 \text{ mm}^2$ $\quad$ 4 ϕ 22

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{550}{2} = 275 \text{ mm} \\ t_b = \frac{spacing}{12} = \frac{5000}{12} = 416.6 \text{ mm} \\ t - \frac{L_c}{3} = 550 - \frac{1500}{3} = 50 \text{ mm} \end{array} \right\}$$

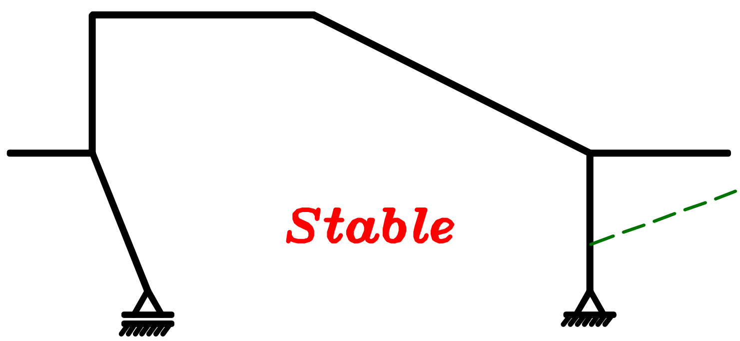
$\boxed{Y = 450 \text{ mm}}$

لن نستطيع أخذ هذا العمود Link member
لان العمود الاخر Roller



لانه فى هذه الحاله اذا وجدت أى HL. Load
لن يوجد أى عمود يستطيع مقاومتها أى يكون Unstable Frame

لذا سنأخذ هذا العمود
لتحمل القوى الافقيه اذا حدثت .



$P = 681.13 \ kN \quad , \quad b = 350 \ mm$

$t = 1300 \ mm$ (the same depth of the beam)

$$\therefore P_{U.L.} = 0.35 A_c F_{cu} + 0.67 A_s F_y$$

$$\therefore 681.13 * 10^3 = 0.35 (350 * 1300)(25) + 0.67 A_s (360)$$

$$\therefore A_s = -13682 \ mm^2 = -(Ve) \ Value$$

$$\therefore A_{s_{total}} = A_{s_{min.}} = \frac{0.8}{100} * 350 * 1300 = 3640 \ mm^2$$

$$\therefore A_s = A_s` = \frac{A_{s \ min.}}{2} = \frac{3640}{2} = 1820 \ mm^2$$

5 ϕ 22

Check Shear.

- Allowable shear stress.

- $q_{cu} = 0.24 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.24 \sqrt{\frac{25}{1.5}} = 0.98 \ N\backslash mm^2$

- $q_{max.} = 0.7 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.7 \sqrt{\frac{25}{1.5}} = 2.85 \ N\backslash mm^2$

Sec. ① $Q = 429.8$ kN

$\therefore$ Actual shear stress. $= q_U = \frac{Q}{b\,d} = \frac{429.8 * 10^3}{350 * 1200} = 1.02 \ N\backslash mm^2$

$\therefore q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5 \phi 8 \backslash m$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n A_s (F_y \backslash \delta_s)}{b \ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3 \ mm^2$

$1.02 - \frac{0.98}{2} = \frac{2 * 50.3 \ (240 \backslash 1.15)}{350 * S} \longrightarrow S = 113.1 \ mm > 100 \ mm$ $\therefore$ o.k.

$\therefore$ No. of stirrups$\backslash m` = \frac{1000}{S} = \frac{1000}{113.1} = 8.84 = 9.0$

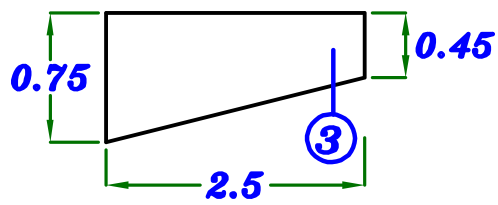
$\therefore$ Use Stirrups $\boxed{9 \phi 8 \backslash m}$ 2 branches

Sec. ② $Q = 263.67$ kN

$\therefore$ Actual shear stress. $= q_U = \frac{Q}{b\,d} = \frac{263.67 * 10^3}{350 * 1200} = 0.627 \ N\backslash mm^2$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m`}$ 2 branches

Sec. ③ $Q = 120$ kN



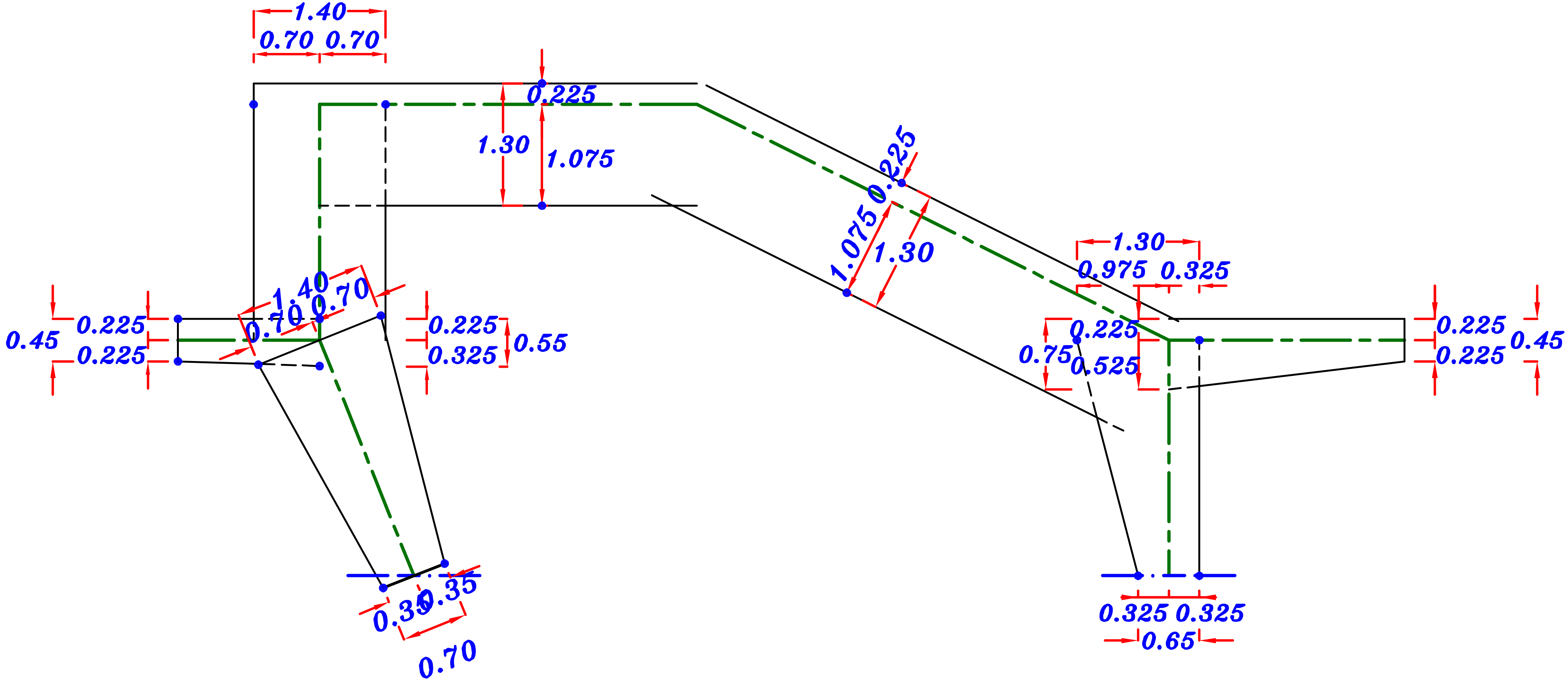
$\therefore$ Actual shear stress. =

$q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b \ d^2}$

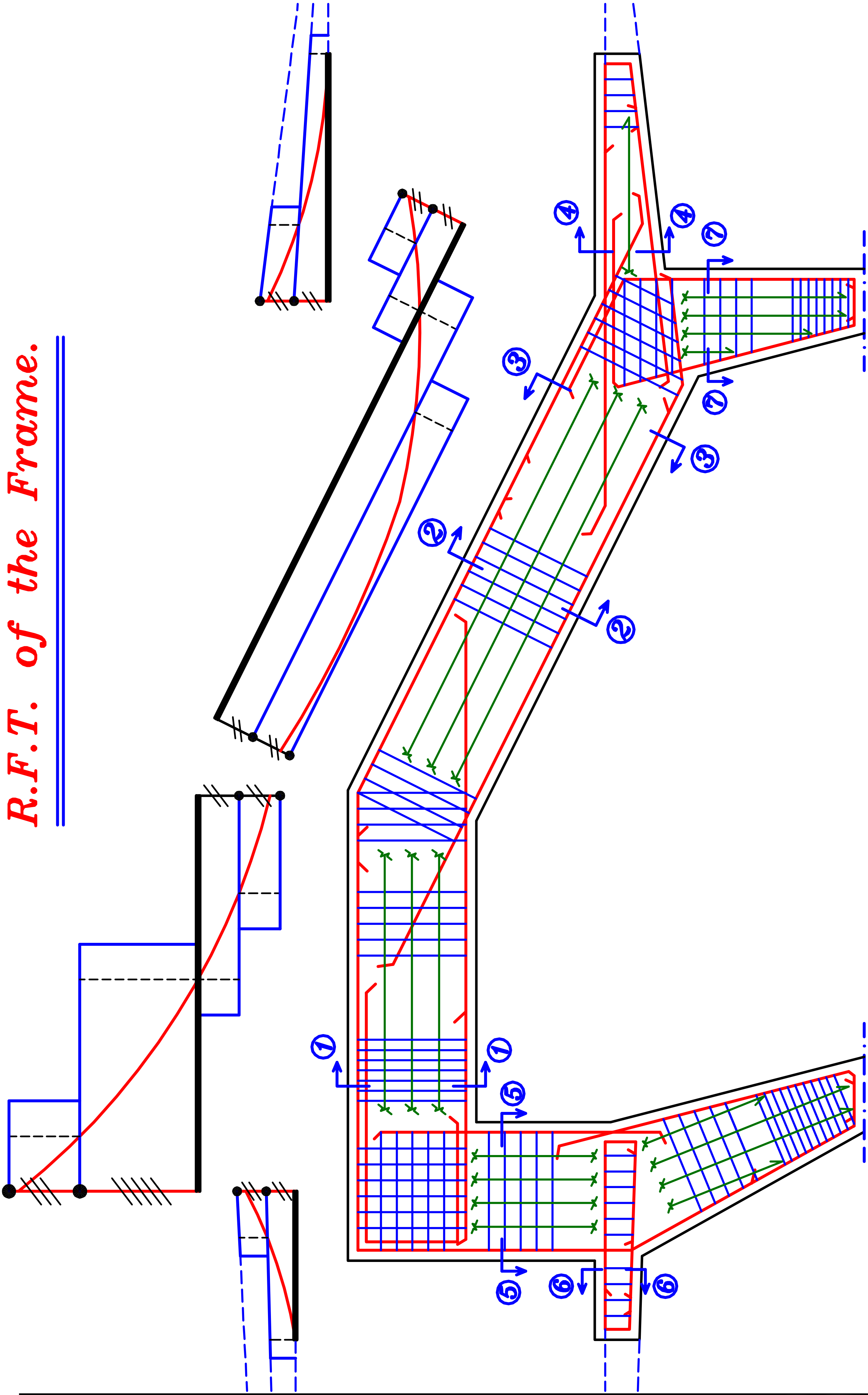
$= \frac{120.0 * 10^3}{350 * 400} - ZERO = 0.857 \ N\backslash mm^2$

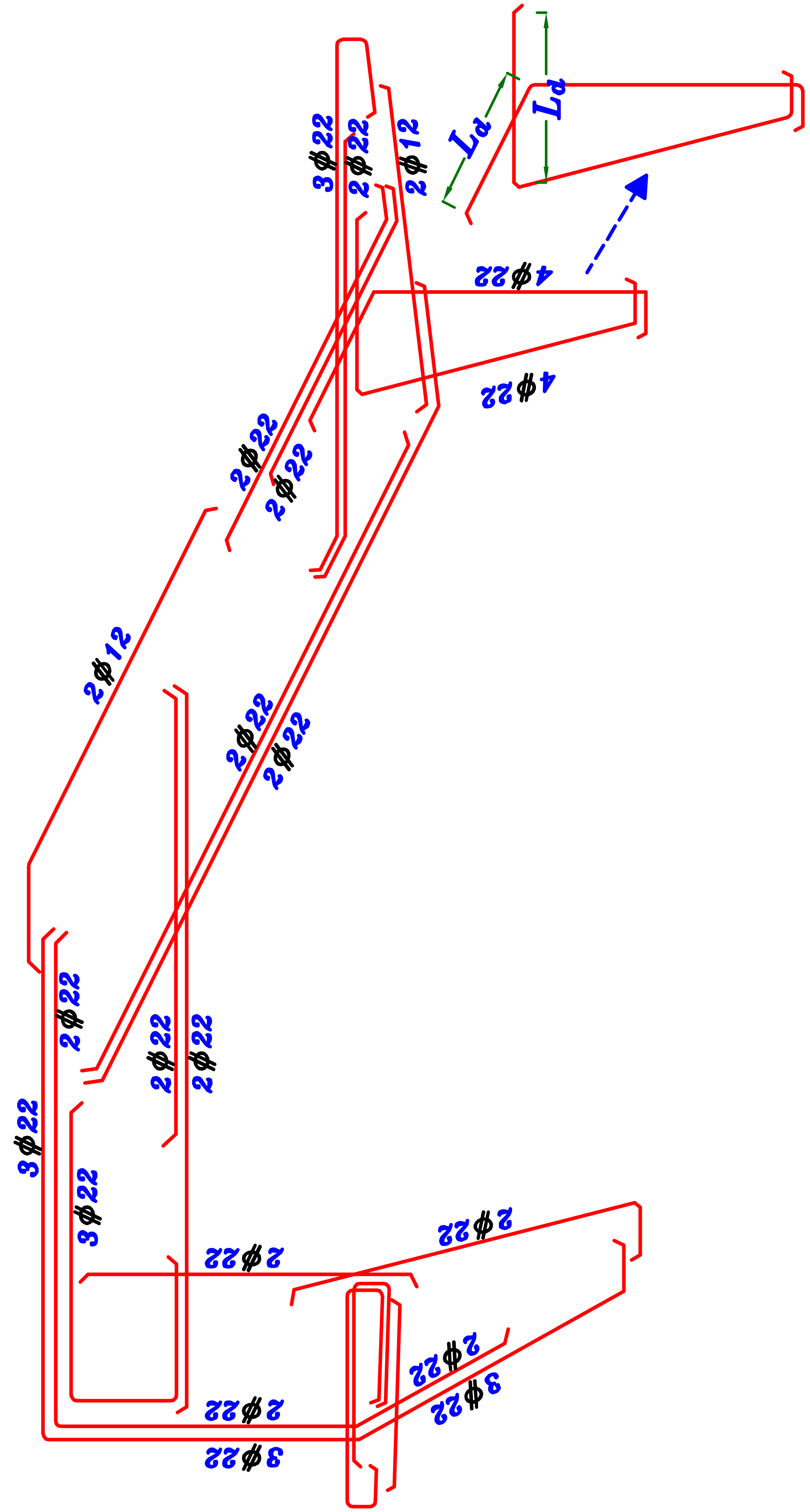
$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m`}$

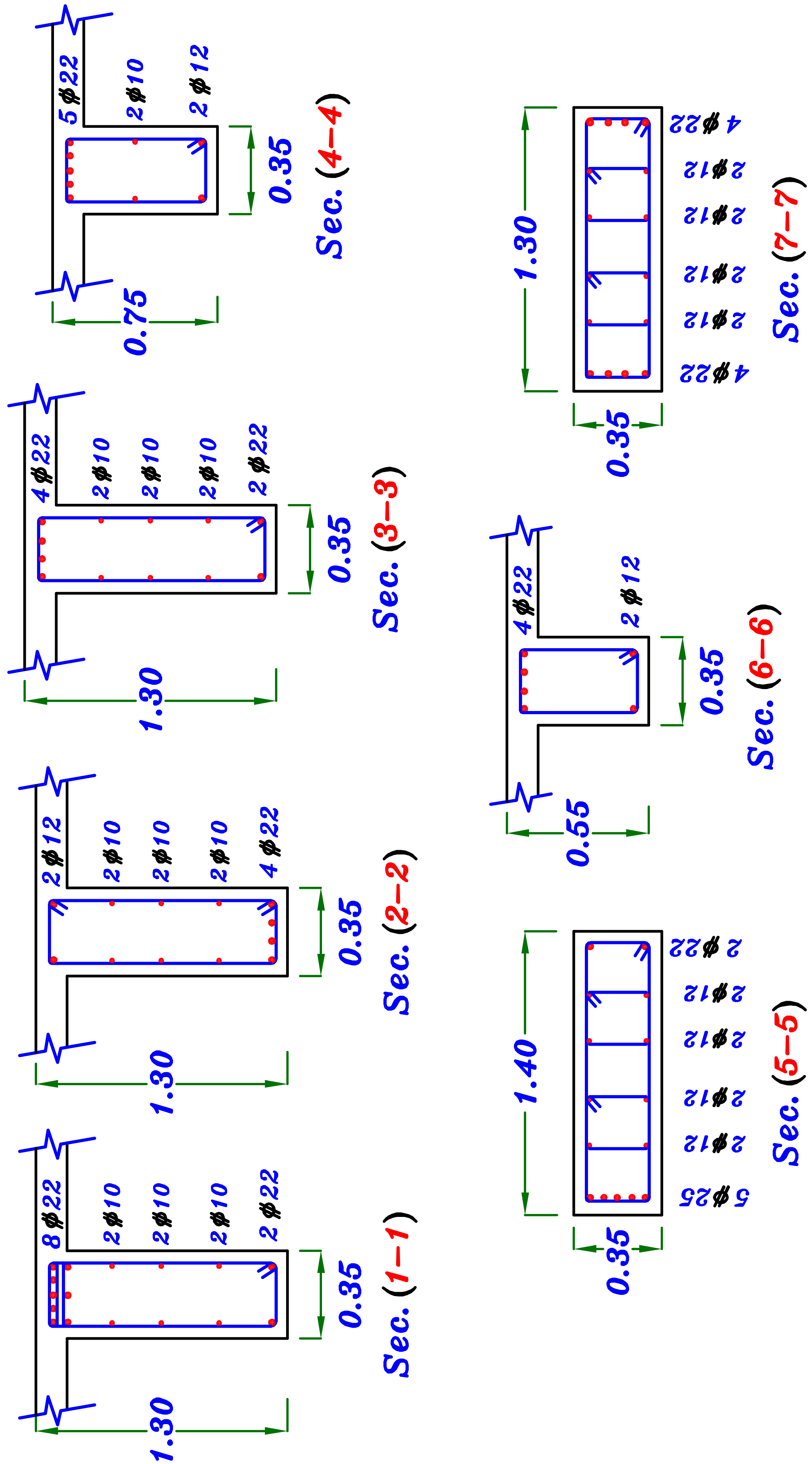
5- *Draw details of reinforcement For Frame (F) in elevation to scale 1:50 and cross sections to scale 1:25*



R.F.T. of the Frame.




Ld
Ld
3ϕ22
2ϕ22
2ϕ12
4ϕ22
4ϕ22
2ϕ22
2ϕ22
2ϕ12
2ϕ22
2ϕ22
2ϕ22
2ϕ22
2ϕ22
3ϕ22
3ϕ22
2ϕ22
2ϕ22
2ϕ22
2ϕ22
3ϕ22
3ϕ22



Sec. (1-1)
8 φ22
2 φ10
2 φ10
2 φ10
2 φ22
0.35
1.30
Sec. (2-2)
2 φ12
2 φ10
2 φ10
2 φ10
4 φ22
0.35
1.30
Sec. (3-3)
4 φ22
2 φ10
2 φ10
2 φ10
2 φ22
0.35
1.30
Sec. (4-4)
5 φ22
2 φ10
2 φ12
0.35
0.75
Sec. (5-5)
2 φ22
2 φ12
2 φ12
2 φ12
2 φ12
5 φ25
1.40
0.35
Sec. (6-6)
4 φ22
2 φ12
0.35
0.55
Sec. (7-7)
4 φ22
2 φ12
2 φ12
2 φ12
2 φ12
4 φ22
1.30
0.35

Example.

The reinforced concrete hall, Whose cross-section is shown in Fig. (1), Consists of a system of solid slabs and secondary beams supported on successive Frame (F) spaced at 6.0 m. **It is required to:**

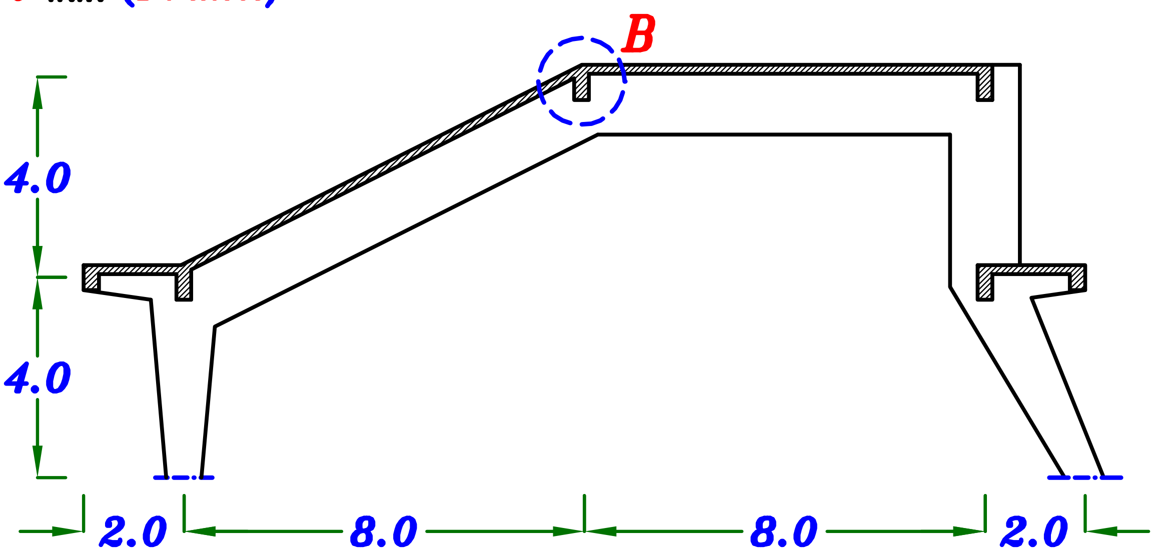
a- Draw the N.F.D , S.F.D & B.M.D For the intermediate Frame (F) using the given working total loads.

b - Design the critical sections of Frame (F) to satisfy the internal Forces requirements, using the limit state design method (LSDM)

c - Draw the details of reinforcement For intermediate Frame (F) in elevation to scale 1:50 and cross sections to scale 1:25 . Curtailment of bars using the moment of resistance diagram is required.

Data:

- Concrete characteristic strength $F_{cu} = 30$ N/mm^2
- Steel used is St. 400/600
- $b = 400$ mm (Frame)



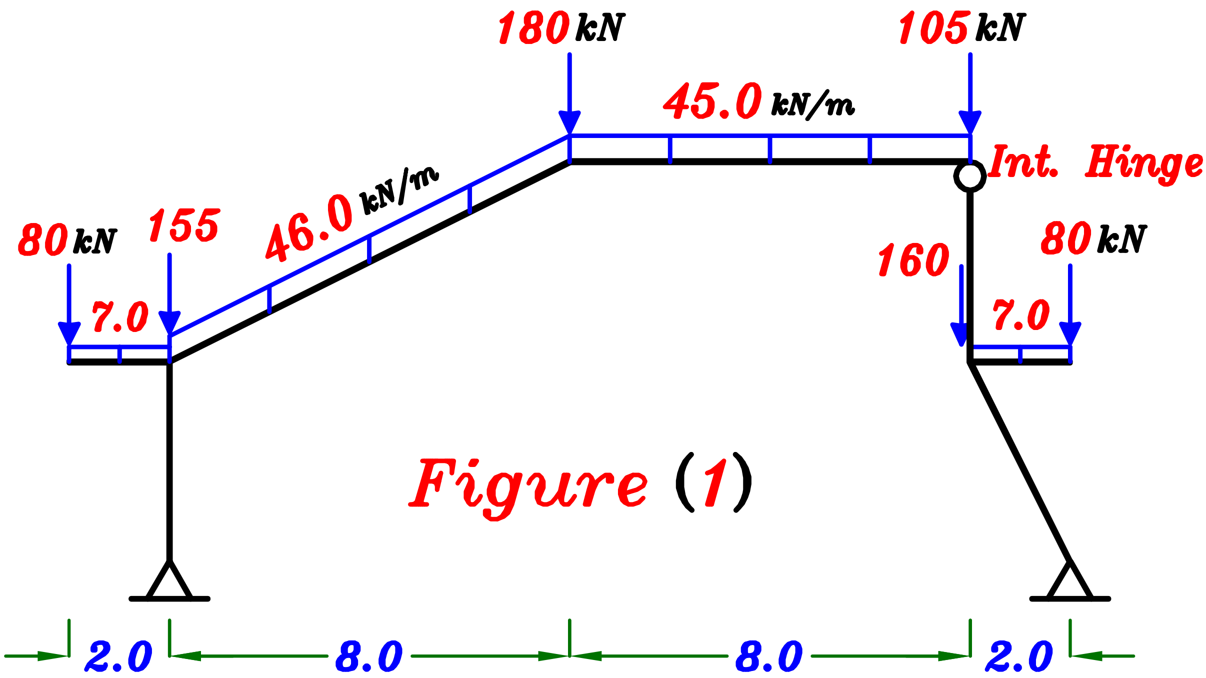
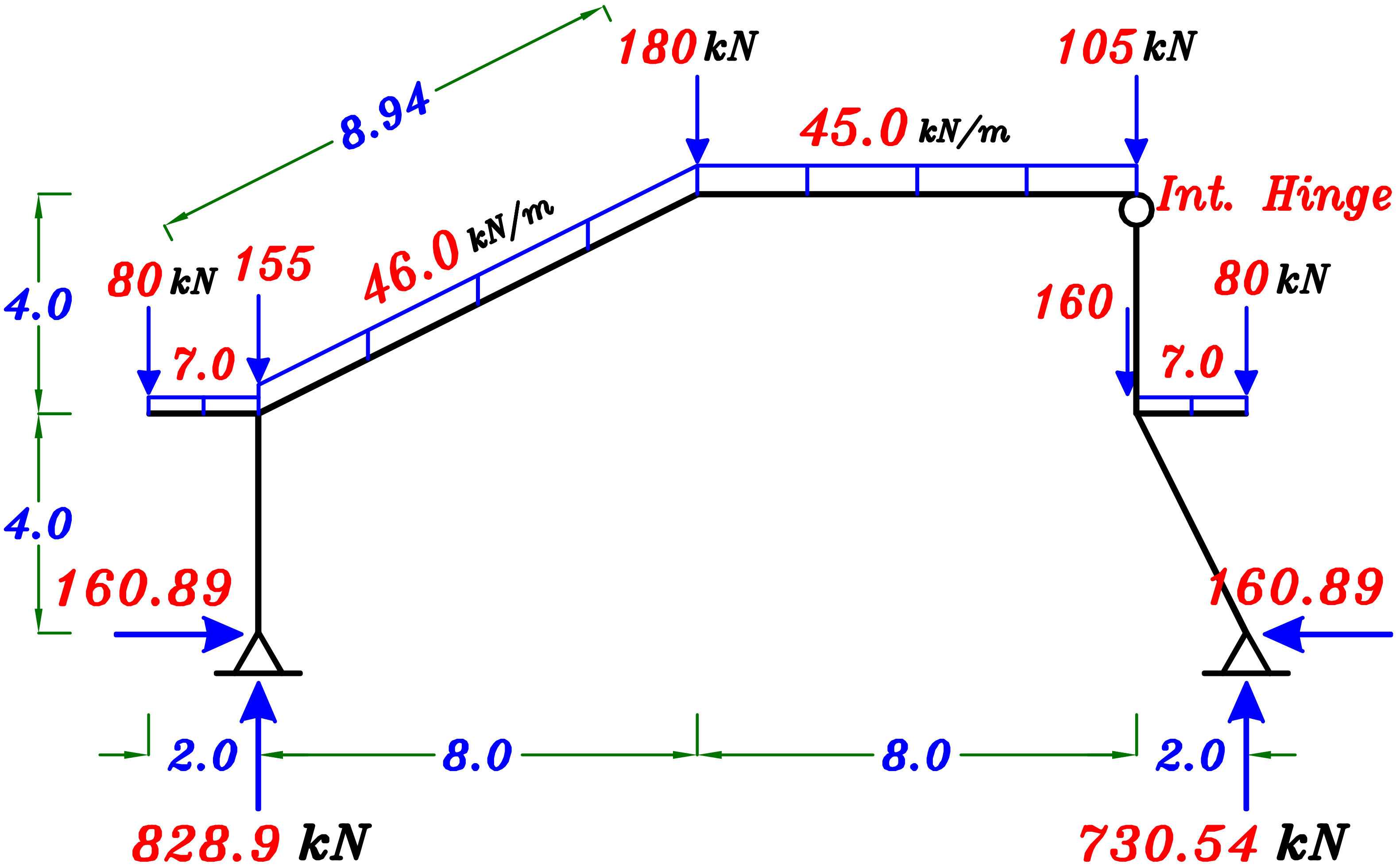
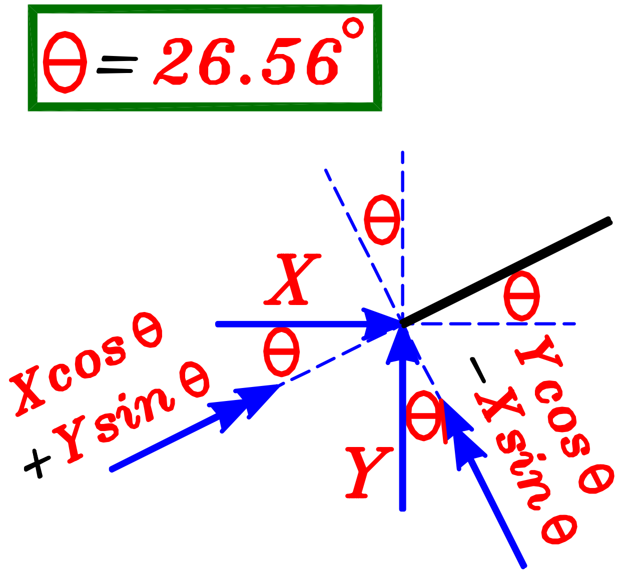
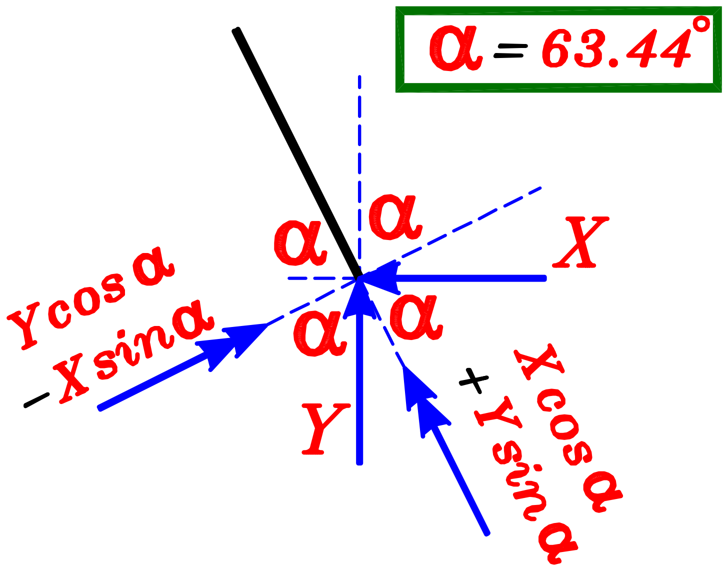


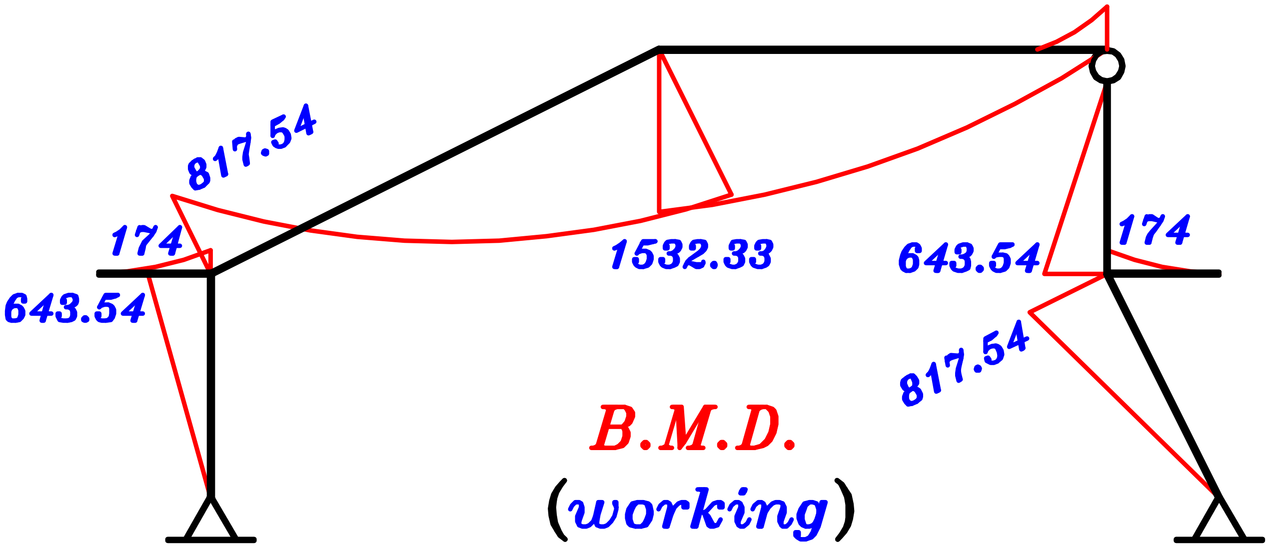
Figure (1)

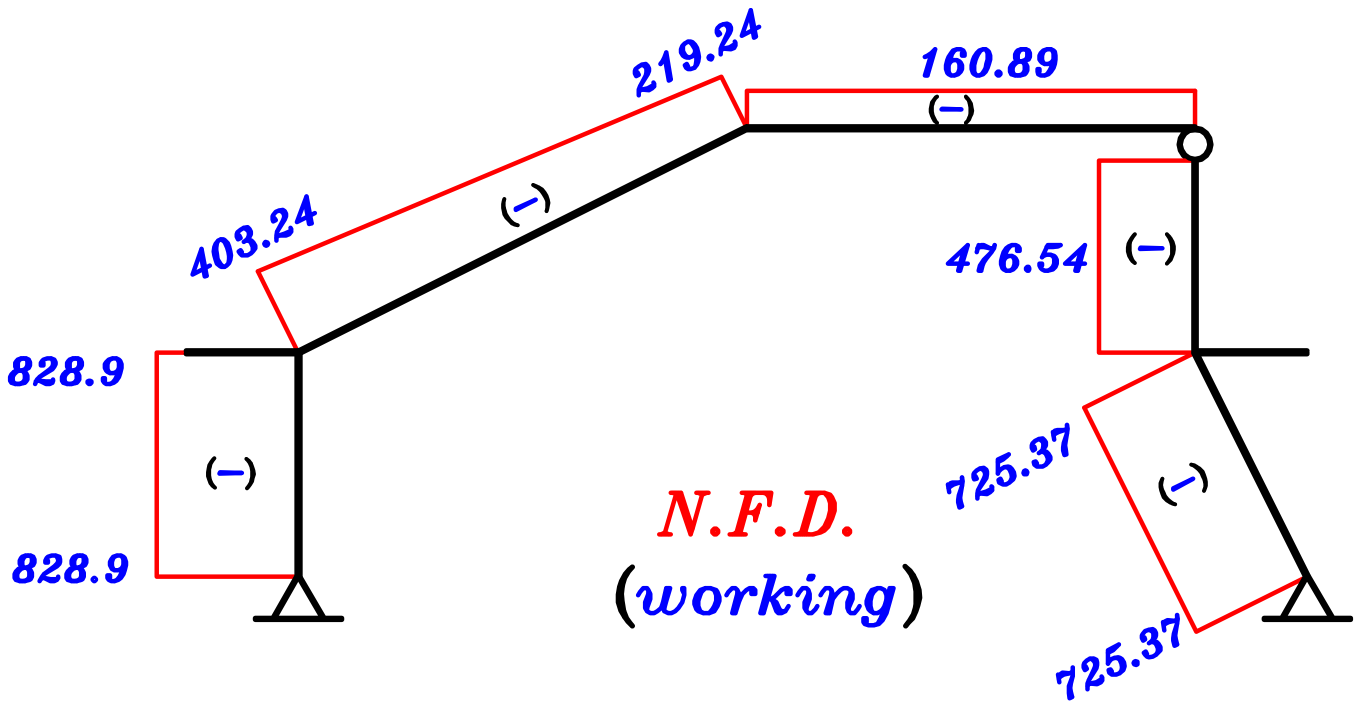
a – Draw the N.F.D , S.F.D & B.M.D For the intermediate Frame (F) using the given working total loads.

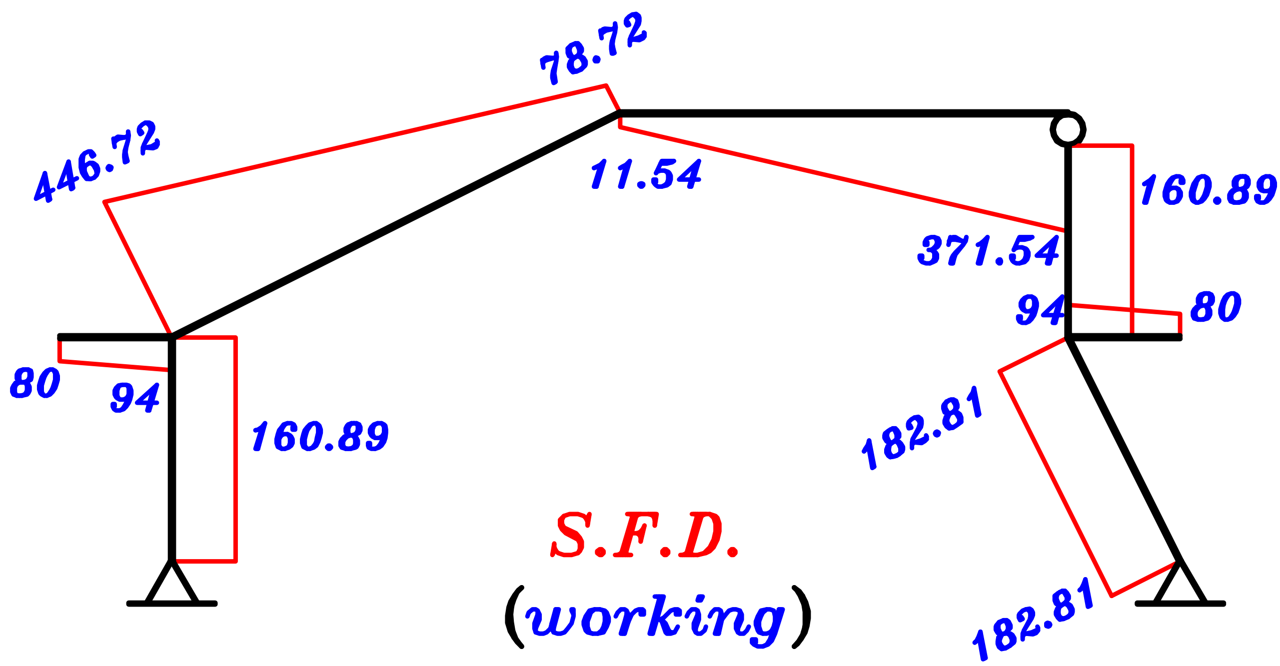


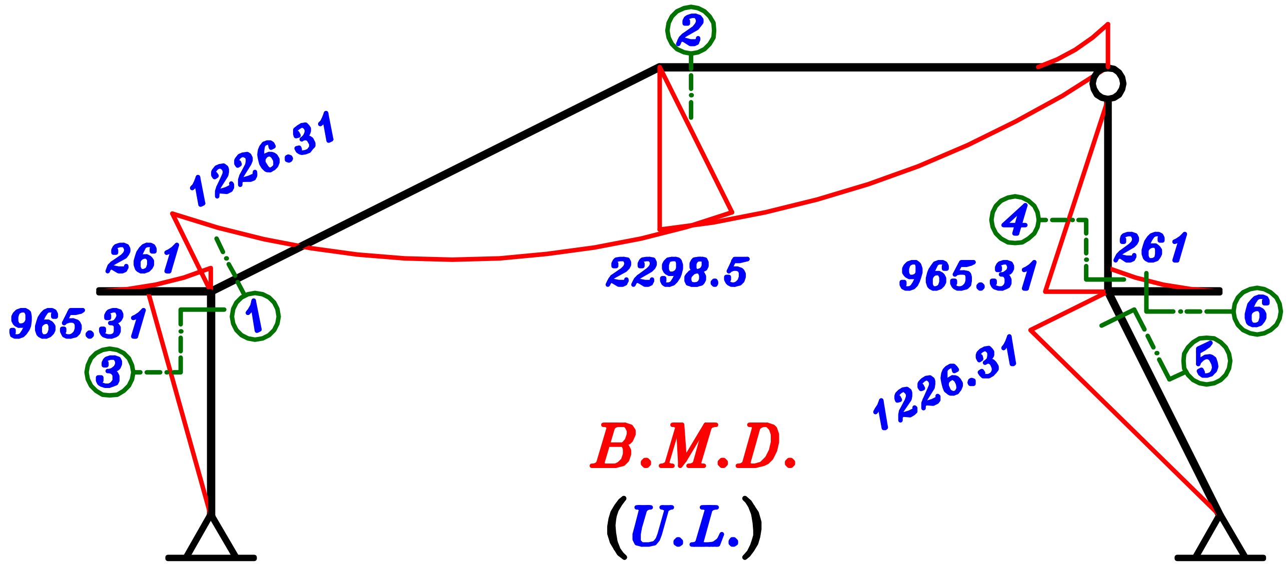




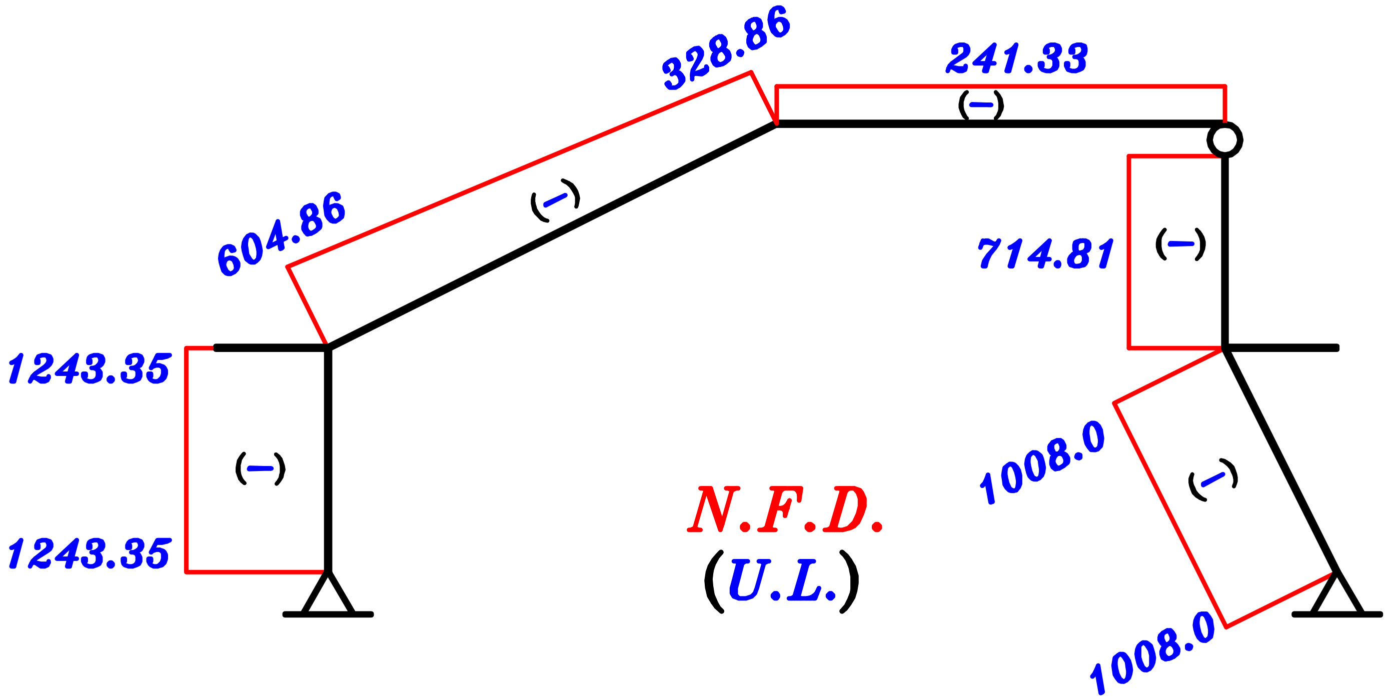

817.54
174
643.54
1532.33
643.54
174
817.54
B.M.D.
(working)


219.24
160.89
(–)
403.24
(–)
476.54
(–)
828.9
(–)
828.9
725.37
(–)
N.F.D.
(working)
725.37

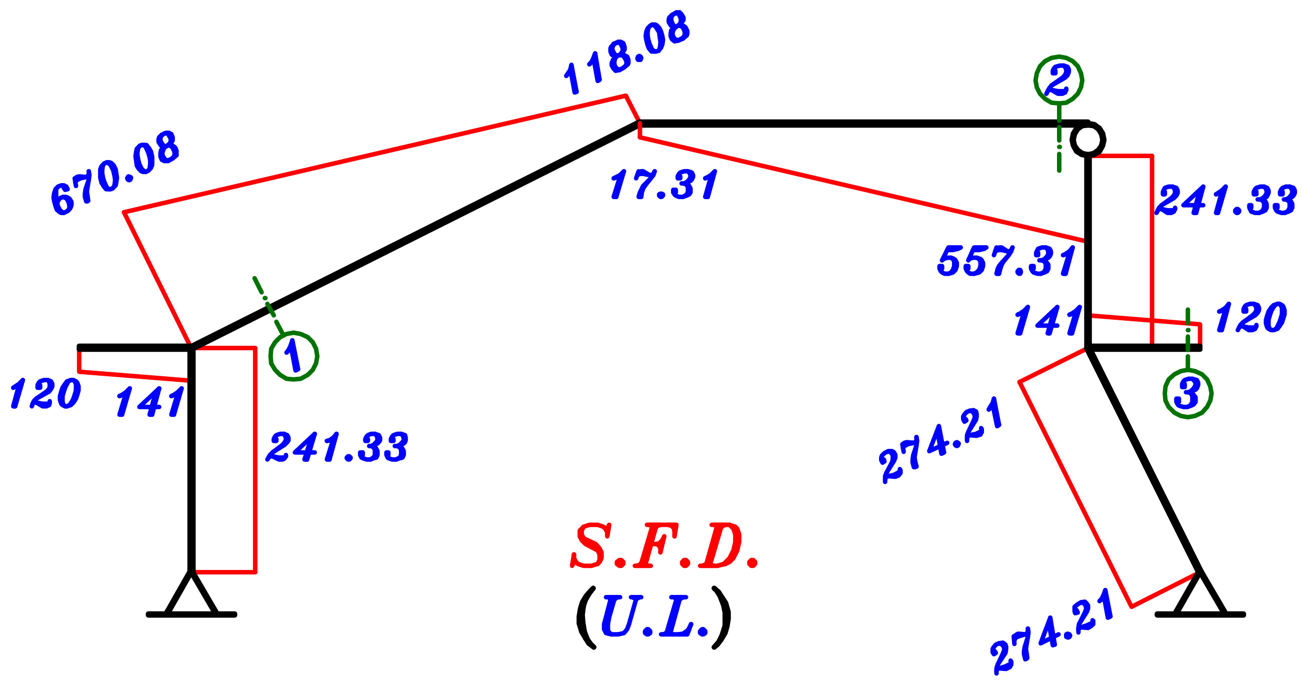

78.72
446.72
11.54
160.89
371.54
94
80
80
94
160.89
182.81
S.F.D.
(working)
182.81



1226.31
261
965.31
2298.5
965.31
261
1226.31
B.M.D.
(U.L.)



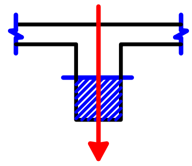
328.86
241.33
604.86
714.81
1243.35
1243.35
1008.0
1008.0
N.F.D.
(U.L.)



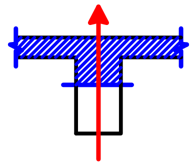
118.08
670.08
17.31
241.33
557.31
141
120
120
141
241.33
274.21
274.21
S.F.D.
(U.L.)

b – Design the critical sections of Frame (F) to satisfy the internal Forces requirements, using the limit state design method (LSDM)

Sec. ① $M_{U.L.} = 1226.31$ kN.m R–Sec.



Sec. ② $M_{U.L.} = 2298.5$ kN.m T–Sec.



$\because M_T < 2\,M_R \quad \therefore$ Design R–Sec. First.

Sec. ① $M = 1226.31$ kN.m , $P = 604.86$ kN , $b = 400$ mm

$$d_o = 3.5\sqrt{\frac{1226.31*10^6}{30*400}} = 1118.86 \text{ mm} \quad (\text{as R–Sec.})$$

$$d = (1.1 \rightarrow 1.3)\,d_o = (1.1 \rightarrow 1.3)\,(1118.86) = (1230.74 \rightarrow 1454.5) \text{ mm}$$

Take $d = 1300$ mm , $t = 1300 + 100 = 1400$ mm

Check $\dfrac{P}{F_{cu}\,b\,t} = \dfrac{604.86*10^3}{30*400*1400} = 0.036 < 0.04 \quad \therefore$ (neglect P)

$\therefore$ Take $d = d_o = 1118.86$ mm $\therefore$ Take $\boxed{d = 1200 \text{ mm}}$, $\boxed{t = 1300 \text{ mm}}$

$\because C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{1226.31*10^6}{0.780*400*1118.86} = 3512.9 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 3512.9$ mm^2

$$\mu_{min.}\,b\,d = \left(0.225*\frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225*\frac{\sqrt{30}}{400}\right) 400*1200 = 1478 \text{ mm}^2$$

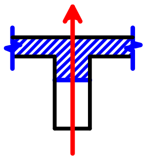
$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3512.9$ mm^2 $\quad 8\phi 25$

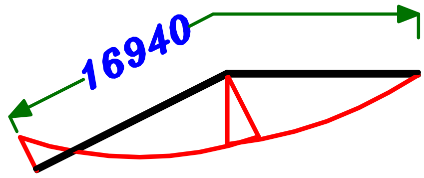
$$\therefore n = \frac{b-25}{\phi+25} = \frac{400-25}{25+25} = 7.50 = 7.0 \text{ bars}$$

Sec. ② $M = 2298.5$ kN.m , $P = 241.33$ kN , $b = 400$ mm

Take $d = 1.20$ m (The same d of Sec. ①)

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{241.33 * 10^3}{30 * 400 * 1300} = 0.0154 < 0.04 \quad \therefore$ (neglect P)

Designed as T–Sec. 



$$B = \begin{cases} C.L.-C.L. = Spacing = 6.0\,m = 6000 \text{ mm} \\ 16\, t_s + b = 16 * 180 + 400 = 3280 \text{ mm} \\ K\dfrac{L}{5} + b = 0.8 * \dfrac{16940}{5} + 400 = 3110 \text{ mm} \end{cases} \qquad \boxed{B = 3110 \text{ mm}}$$

$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, B}} \quad \therefore 1200 = C_1 \sqrt{\frac{2298.5 * 10^6}{30 * 3110}} \longrightarrow C_1 = 7.64 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{2298.5 * 10^6}{0.826 * 400 * 1200} = 5797.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 5797.2$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 400 * 1200 = 1478 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 5797.2$ mm^2 $\quad$ 12∅25

Sec. ③ R–Sec. $M = 965.31$ kN.m , $P = 1243.35$ kN

$$d_o = 3.5 \sqrt{\frac{965.31 * 10^6}{30 * 400}} = 992.7 \text{ mm} \qquad (\text{as R–Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(992.7) = (1092 \rightarrow 1290.5) \text{ mm}$$

$\therefore$ Take $\boxed{d = 1100 \text{ mm}}$, $\boxed{t = 1200 \text{ mm}}$

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{1243.35 * 10^3}{30 * 400 * 1200} = 0.086 > 0.04 \quad (\text{Don't neglect } P)$$

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{965.31}{1243.35} = 0.77 \text{ m} \qquad \therefore \frac{e}{t} = \frac{0.77}{1.20} = 0.64 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 0.77 + \frac{1.20}{2} - 0.10 = 1.27 \text{ m}$$

$$M_s = P * e_s = 1243.35 * 1.27 = 1579.0 \text{ kN.m}$$

$$\therefore 1100 = C_1 \sqrt{\frac{1579.0 * 10^6}{30 * 400}} \longrightarrow C_1 = 3.03 \longrightarrow J = 0.747$$

$$\because A_s = \frac{M_s}{J F_y d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{1579.0 * 10^6}{0.747 * 400 * 1100} - \frac{1243.35 * 10^3}{(400 \backslash 1.15)} = 1229 \text{ mm}^2$$

<u>Check $A_{s_{min.}}$</u> $\qquad A_{s_{req.}} = 1229 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 400 * 1100 = 1355.6 \text{ mm}^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \dfrac{\sqrt{30}}{400}\right) 400 * 1100 = 1355.6 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 1229 = 1597.7 \text{ mm}^2 \end{array}\right\} = 1355.6 \text{ mm}^2$$

الأقل

3 ϕ 25

Sec. ④ R-Sec. $M = 965.31$ kN.m , $P = 714.81$ kN

$$d_o = 3.5\sqrt{\frac{965.31 * 10^6}{30 * 400}} = 992.7 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(992.7) = (1092 \rightarrow 1290.5)\text{ mm}$$

$\therefore$ Take $\boxed{d = 1100 \text{ mm}}$, $\boxed{t = 1200 \text{ mm}}$

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{714.81 * 10^3}{30 * 400 * 1200} = 0.049 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{965.31}{714.81} = 1.35 \text{ m} \quad \therefore \frac{e}{t} = \frac{1.35}{1.20} = 1.12 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.35 + \frac{1.20}{2} - 0.10 = 1.85 \text{ m}$$

$$M_s = P * e_s = 714.81 * 1.85 = 1322.4 \text{ kN.m}$$

$$\therefore 1100 = C_1\sqrt{\frac{1322.4 * 10^6}{30 * 400}} \rightarrow C_1 = 3.31 \rightarrow J = 0.769$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{1322.4 * 10^6}{0.769 * 400 * 1100} - \frac{714.81 * 10^3}{(400 \backslash 1.15)} = 1853.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 1853.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 400 * 1100 = 1355.6 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1853.2 \text{ mm}^2$ 4 ϕ 25

Stirrup Hangers $\simeq 0.4\, A_s \simeq 0.4 * 1853.2 = 741 \text{ mm}^2$ 2 ϕ 22

Sec. 5 R–Sec. $M = 1226.31$ kN.m , $P = 1008.0$ kN

Take $d = 1.10$ m (The same d of Sec. 4)

Check $\frac{P}{F_{cu}\, b t} = \frac{1008.0 * 10^3}{30 * 400 * 1200} = 0.070 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$e = \frac{M}{P} = \frac{1226.31}{1008.0} = 1.21$ m $\therefore \frac{e}{t} = \frac{1.21}{1.20} = 1.0 > 0.5 \xrightarrow{Use} e_s$

$e_s = e + \frac{t}{2} - c = 1.21 + \frac{1.20}{2} - 0.10 = 1.71$ m

$M_s = P * e_s = 1008.0 * 1.71 = 1723.7$ kN.m

$\therefore 1100 = C_1 \sqrt{\frac{1723.7 * 10^6}{30 * 400}} \longrightarrow C_1 = 2.90 \longrightarrow J = 0.734$

$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$

$$= \frac{1723.7 * 10^6}{0.734 * 400 * 1100} - \frac{1008.0 * 10^3}{(400 \backslash 1.15)} = 2439.2 \text{ mm}^2$$

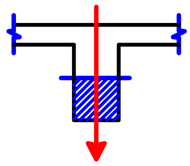
Check $A_{s_{min.}}$ $A_{s_{req.}} = 2439.2 \text{ mm}^2$

$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 400 * 1100 = 1355.6 \text{ mm}^2$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2439.2 \text{ mm}^2$ 5 ϕ 25

Stirrup Hangers $\simeq 0.4\, A_s \simeq 0.4 * 2439.2 = 975.68 \text{ mm}^2$ 2 ϕ 25

Sec. ⑥ $M = 261\ kN.m$, $b = 400\ mm$ R−Sec.



$$d = 3.5\sqrt{\frac{261 * 10^6}{30 * 400}} = 516.1\ mm$$

∴ Take $\boxed{d = 550\ mm}$, $\boxed{t = 600\ mm}$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{261 * 10^6}{0.78 * 400 * 516.1} = 1621\ mm^2$$

Check $A_{s_{min.}}$

$$A_{s_{req.}} = 1621\ mm^2$$

$$\mu_{min.}\ b\ d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\ d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 400 * 550 = 677.8\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1621\ mm^2$$

4 ϕ25

Take $Y = 0.40\ m$

Check Shear.

– Allowable shear stress.

– $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{30}{1.5}} = 1.07\ N\backslash mm^2$

– $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{30}{1.5}} = 3.13\ N\backslash mm^2$

Sec. ① $Q = 670.08\ kN$

∴ Actual shear stress. $= q_U = \frac{Q}{b\ d} = \frac{670.08 * 10^3}{400 * 1200} = 1.39\ N\backslash mm^2$

$\therefore\ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8 \backslash m'$

$\therefore$ Use $q_s = q_u - \dfrac{q_{cu}}{2} = \dfrac{n A_s (F_y \backslash \delta_s)}{b\ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.39 - \frac{1.07}{2} = \frac{2 * 50.3\ (240 \backslash 1.15)}{400 * S} \longrightarrow S = 61.3\ mm < 100\ mm$$

* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$$1.39 - \frac{1.07}{2} = \frac{2 * 78.5\ (240 \backslash 1.15)}{400 * S} \longrightarrow S = 95.8\ mm < 100\ mm$$

* Take $n = 4$, $\phi\ 8 \longrightarrow A_s = 78.5\ mm^2$

$$1.39 - \frac{1.07}{2} = \frac{4 * 50.3\ (240 \backslash 1.15)}{400 * S} \longrightarrow S = 122.8\ mm > 100\ mm \quad \therefore\ o.k.$$

$\therefore$ No. of stirrups$\backslash m'$ $= \dfrac{1000}{S} = \dfrac{1000}{122.8} = 8.14 = 9.0$

$\therefore$ Use Stirrups $\boxed{9\phi 8 \backslash m'}$ 4 branches

Sec. 2 $Q = 557.31\ kN$

$\therefore$ Actual shear stress. $= q_U = \dfrac{Q}{b\ d} = \dfrac{557.31 * 10^3}{400 * 1200} = 1.16\ N\backslash mm^2$

$\therefore\ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8 \backslash m'$

$\therefore$ Use $q_s = q_u - \dfrac{q_{cu}}{2} = \dfrac{n A_s (F_y \backslash \delta_s)}{b\ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.16 - \frac{1.07}{2} = \frac{2 * 50.3\ (240 \backslash 1.15)}{400 * S} \longrightarrow S = 83.97\ mm < 100\ mm$$

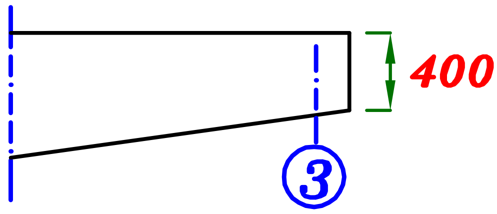
* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$$1.39 - \frac{1.07}{2} = \frac{2 * 78.5\,(240 \backslash 1.15)}{400 * S} \longrightarrow S = 131.0\ mm > 100\ mm \quad \therefore \text{o.k.}$$

$$\therefore \text{No. of stirrups} \backslash m` = \frac{1000}{S} = \frac{1000}{131.0} = 7.63 = 8.0$$

$\therefore$ Use Stirrups $\boxed{8\phi 10 \backslash m`}$ 2 branches

Sec. ③ $Q = 120.0\ kN$



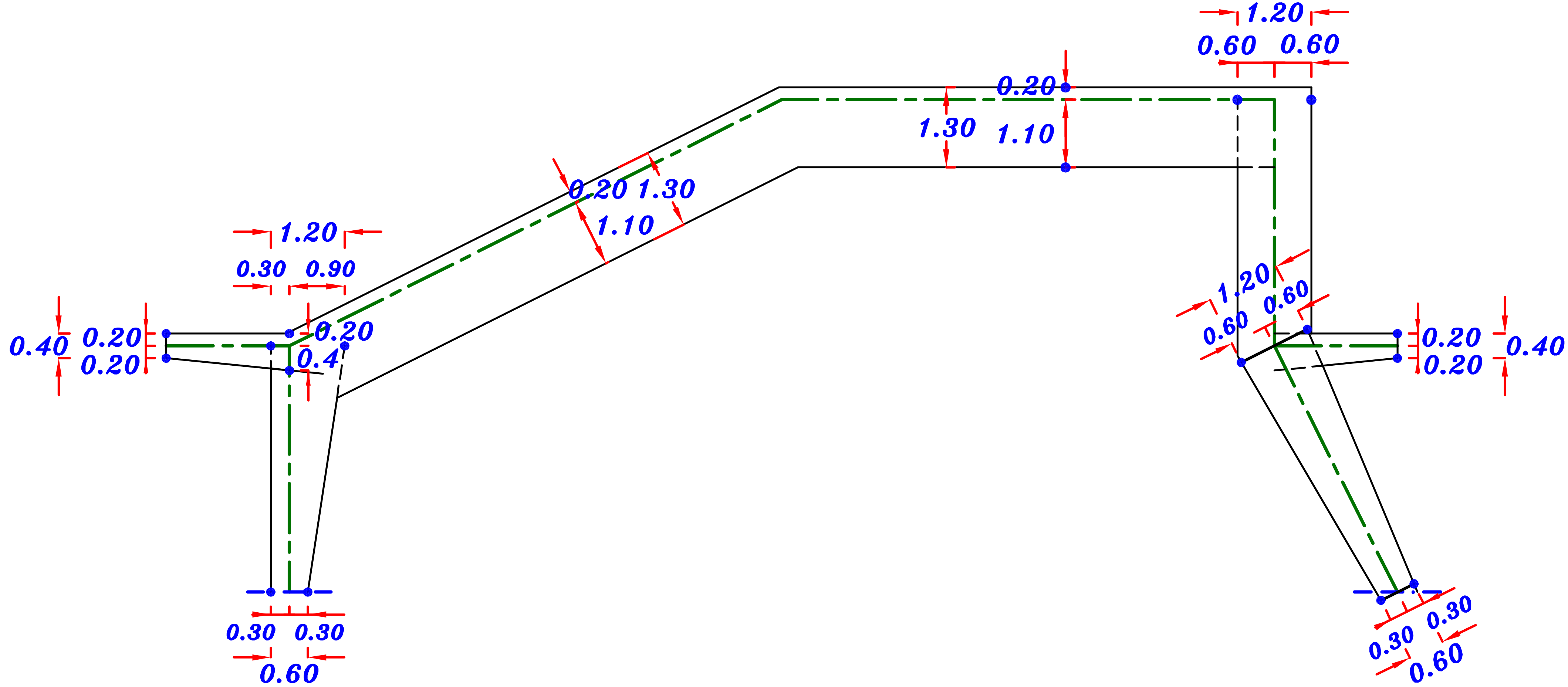
$\therefore$ Actual shear stress. =

$$q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\ d^2}$$

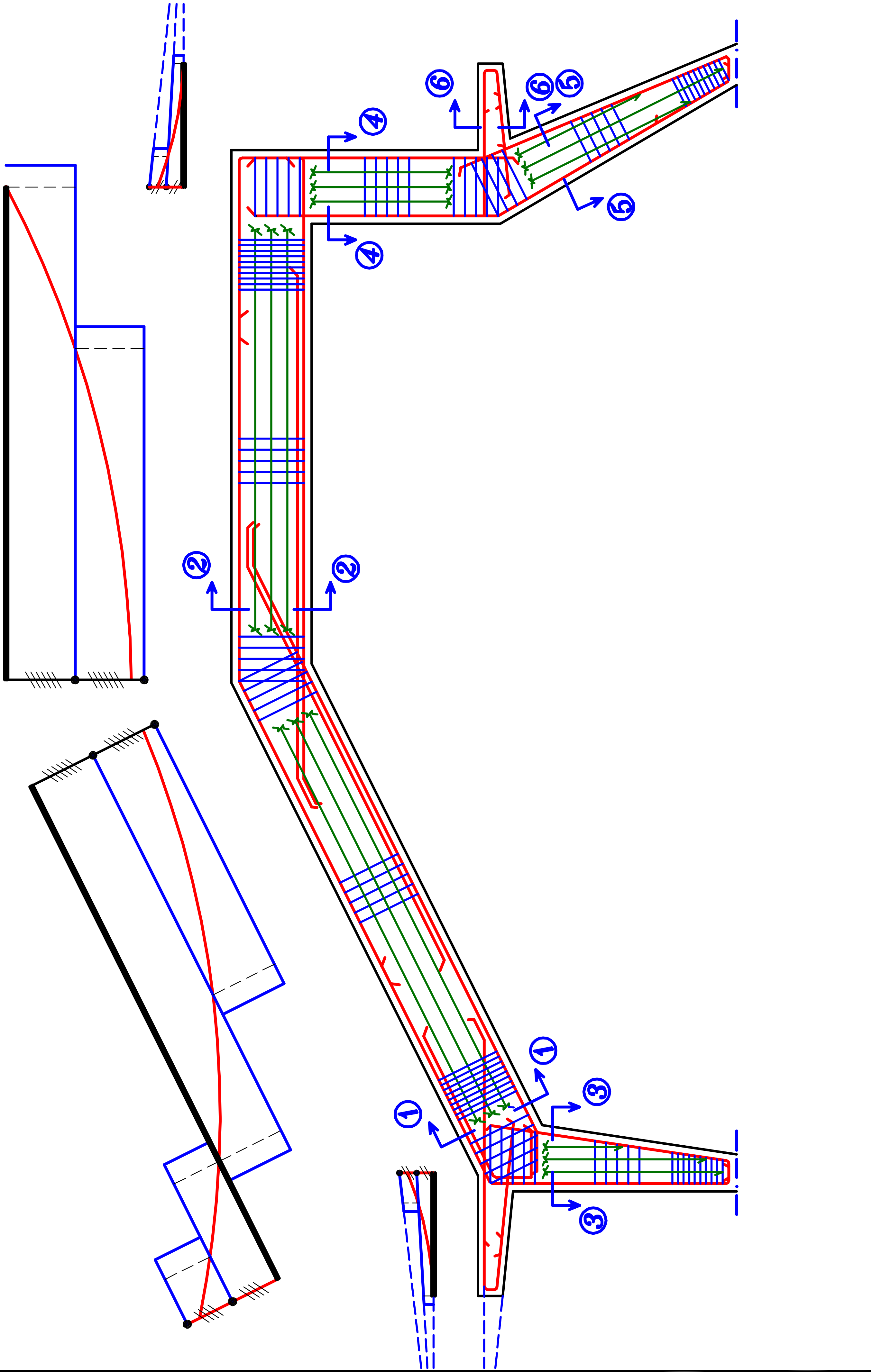
$$= \frac{120.0 * 10^3}{400 * 350} - \text{ZERO} = 0.857\ N\backslash mm^2$$

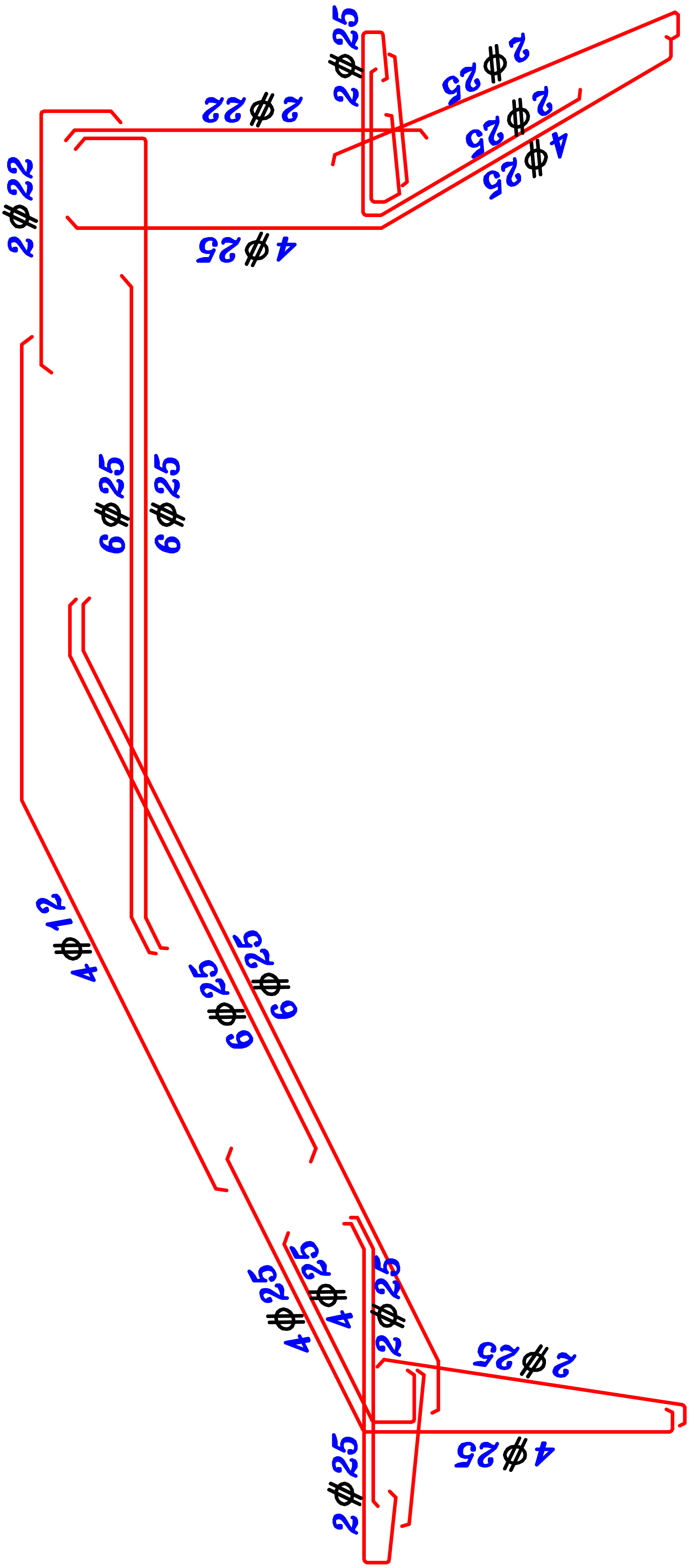
$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m`}$

C – Draw the details of reinforcement For intermediate Frame (F) in elevation to scale 1:50 and cross sections to scale 1:25 . Curtailment of bars using the moment of resistance diagram is required.

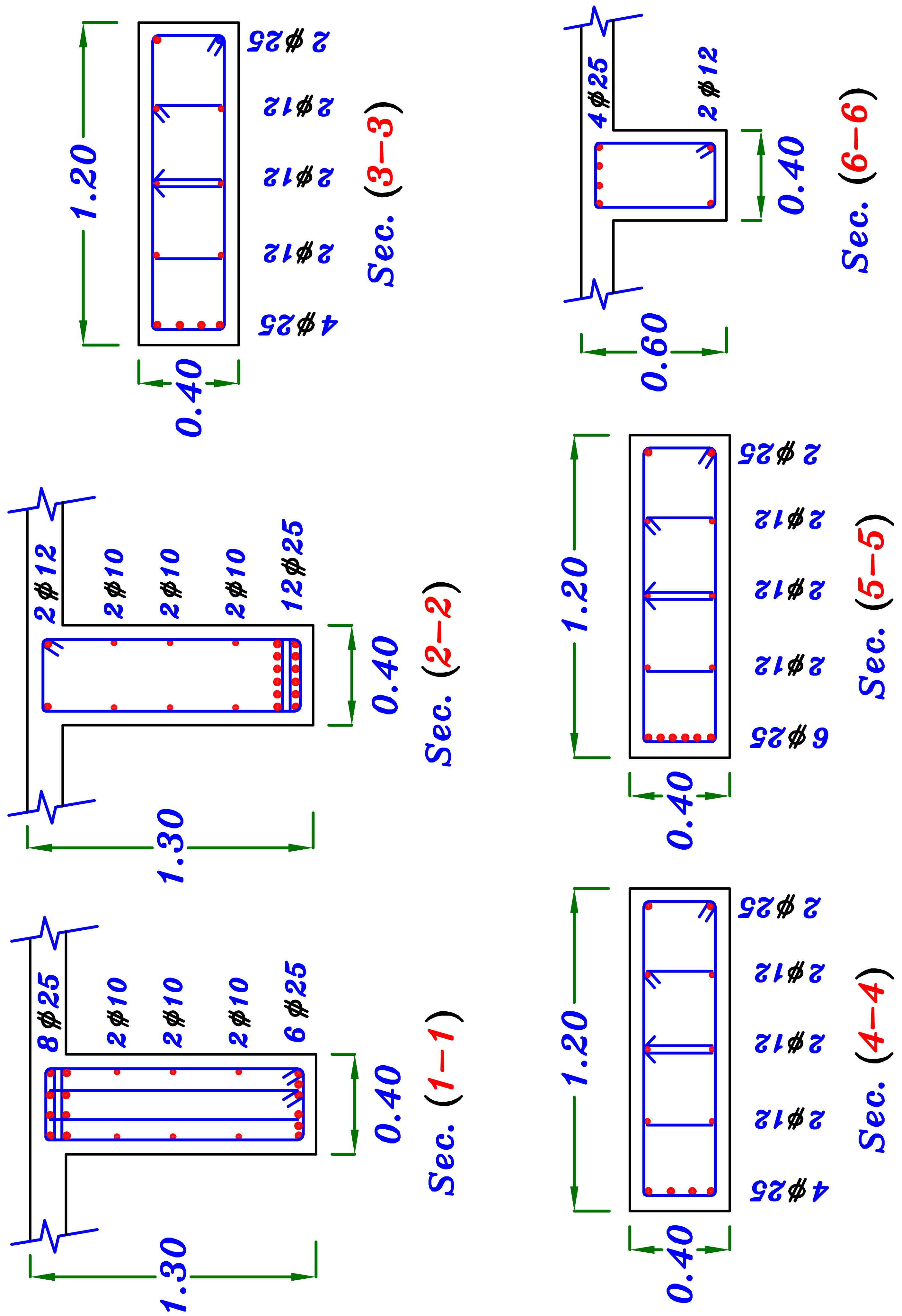


R.F.T. of the Frame.





2Φ25
2Φ25
2Φ22
2Φ25
4Φ25
2Φ22
4Φ25
6Φ25
6Φ25
4Φ12
6Φ25
6Φ25
4Φ25
4Φ25
2Φ25
2Φ25
4Φ25
2Φ25


Sec. (1-1)
1.30
0.40
8 φ25
2 φ10
2 φ10
2 φ10
6 φ25
Sec. (2-2)
1.30
0.40
2 φ12
2 φ10
2 φ10
2 φ10
12 φ25
Sec. (3-3)
1.20
0.40
2 φ25
2 φ12
2 φ12
2 φ12
4 φ25
Sec. (4-4)
1.20
0.40
2 φ25
2 φ12
2 φ12
2 φ12
4 φ25
Sec. (5-5)
1.20
0.40
2 φ25
2 φ12
2 φ12
2 φ12
6 φ25
Sec. (6-6)
0.60
0.40
4 φ25
2 φ12

Example.

Fig. 1A shows a schematic elevation of a main Frame of a roadway concrete bridge. For the given geometrical and loading conditions. **It is required to :**

1 – Draw a sectional elevation showing the concrete dimensions of different structural elements including the Foundation taking into consideration that the Frame are spaced by **6.0 meters**

2 – Draw the **B.M.D. , N.F.D. & S.F.D.** For the main Frame using the given service (working) loads in Fig. **1B**

3 – Design the different concrete sections of the main Frame using ultimate limit design method

4 – Check shear For the horizontal girder only using ultimate limit design method

5 – Draw a sectional elevation (**Scale 1:50**) and cross sections (**Scale 1:25**) showing the reinforcement details of the main Frame using the moment of resistance.

Concrete Grade. C 35 $F_{cu} = 35$ MPa

Steel Grade: st. 52 $F_y = 360$ MPa

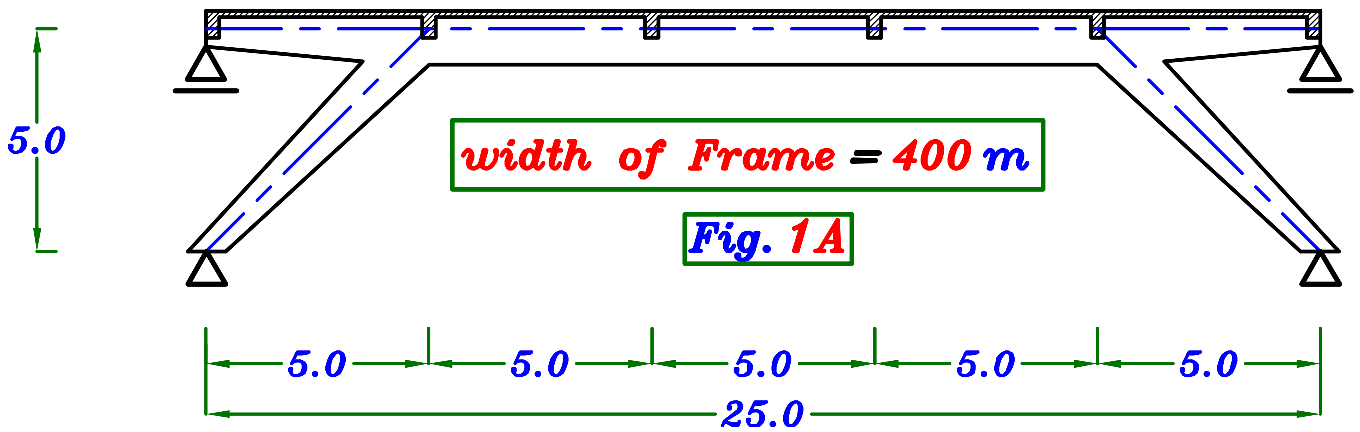


Fig. 1A

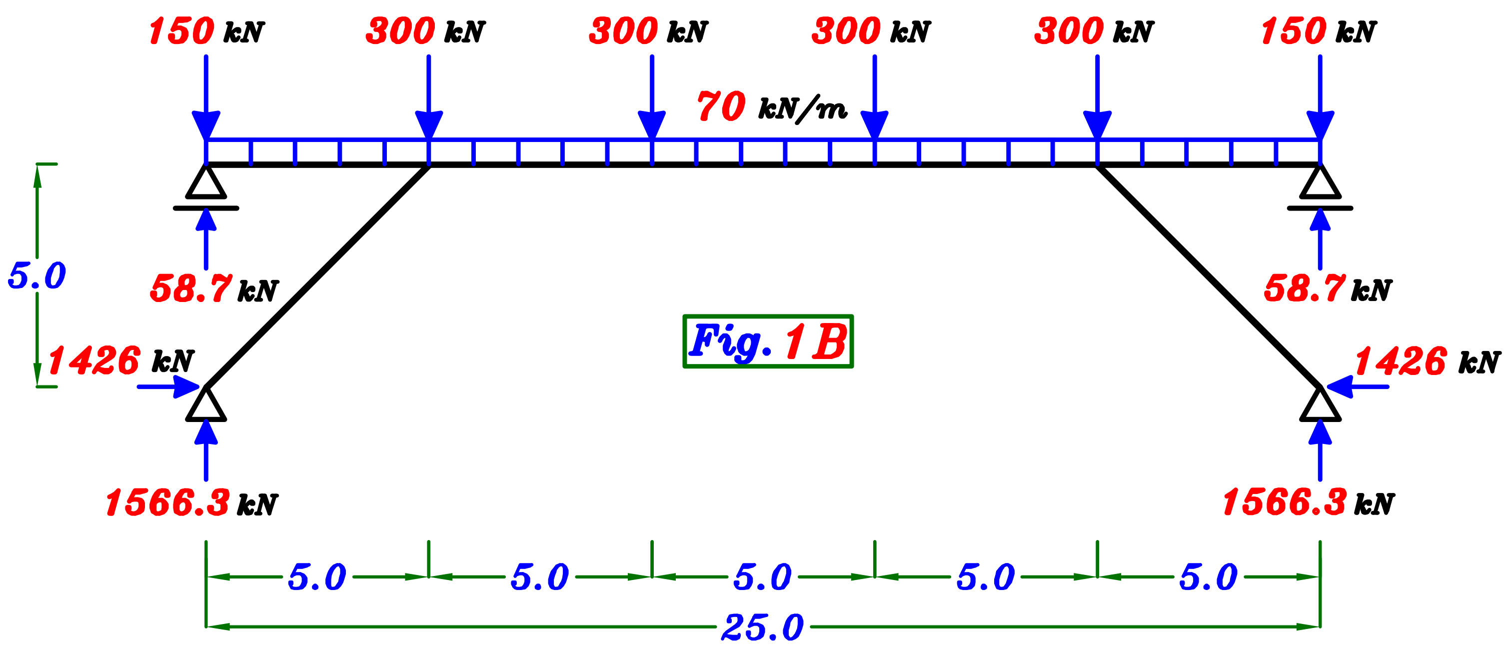
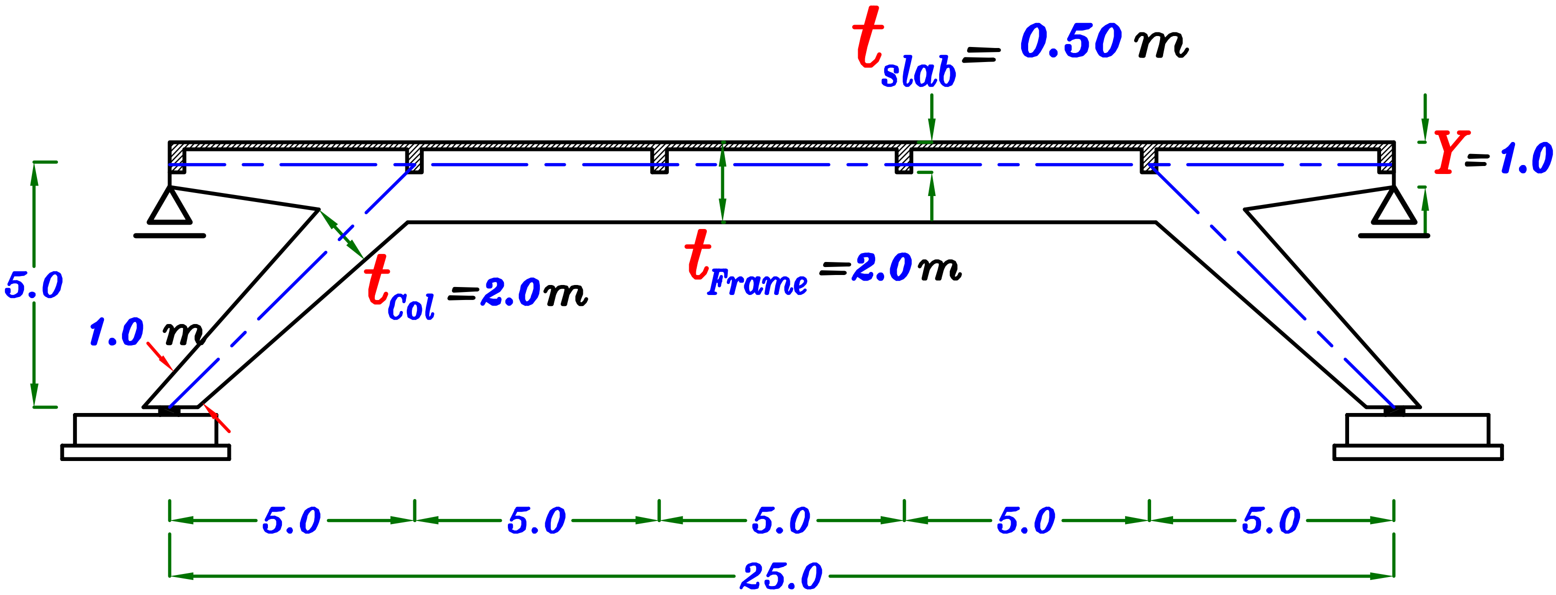


Fig. 1B

Question (1)

1- Draw a sectional elevation showing the concrete dimensions of different structural elements including the Foundation taking into consideration that the Frame are spaced by 6.0 meters



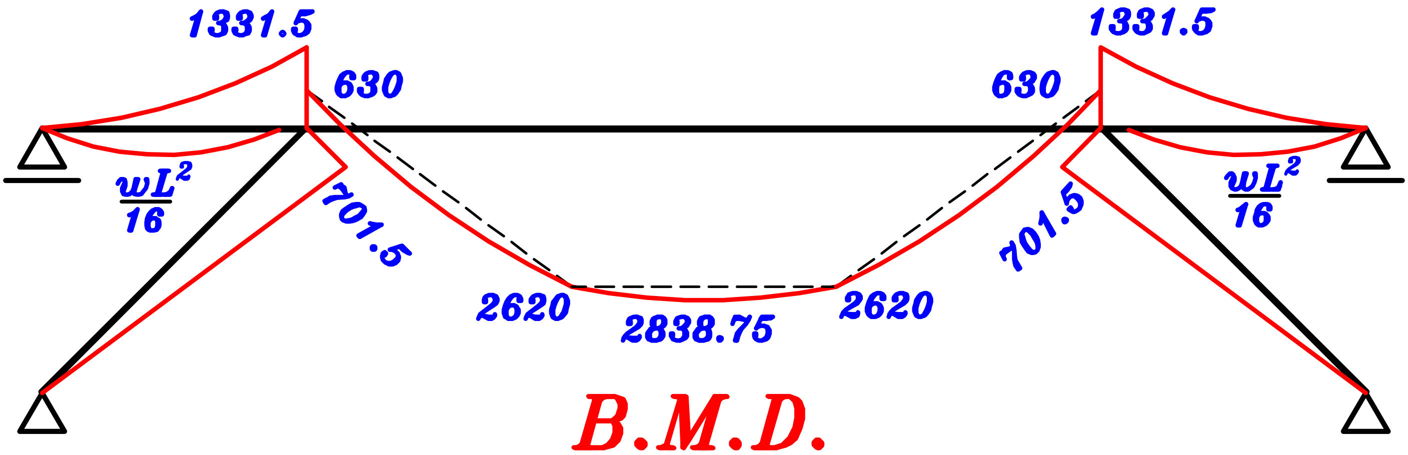
$$t_{Frame} = \frac{L}{12 \rightarrow 14} = \frac{25.0}{12 \rightarrow 14} = (1.78 \rightarrow 2.08)\,m = 2.0\ m$$

$$Y = \frac{t_{Frame}}{2} = 1.0\ m$$

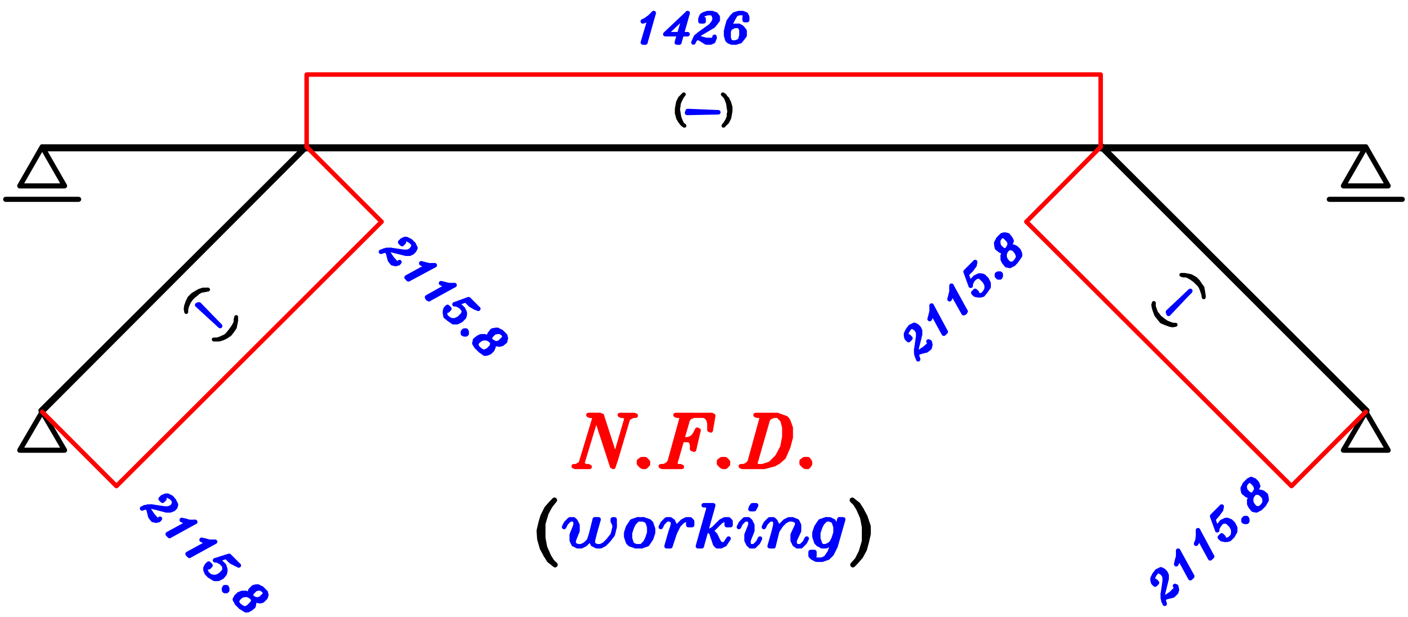
$$t_{col.} = t_{Frame} = 2.0\ m$$

$$t_{secondary\ Beams} = \frac{spacing}{12} = \frac{6.0}{12} = 0.50\ m$$

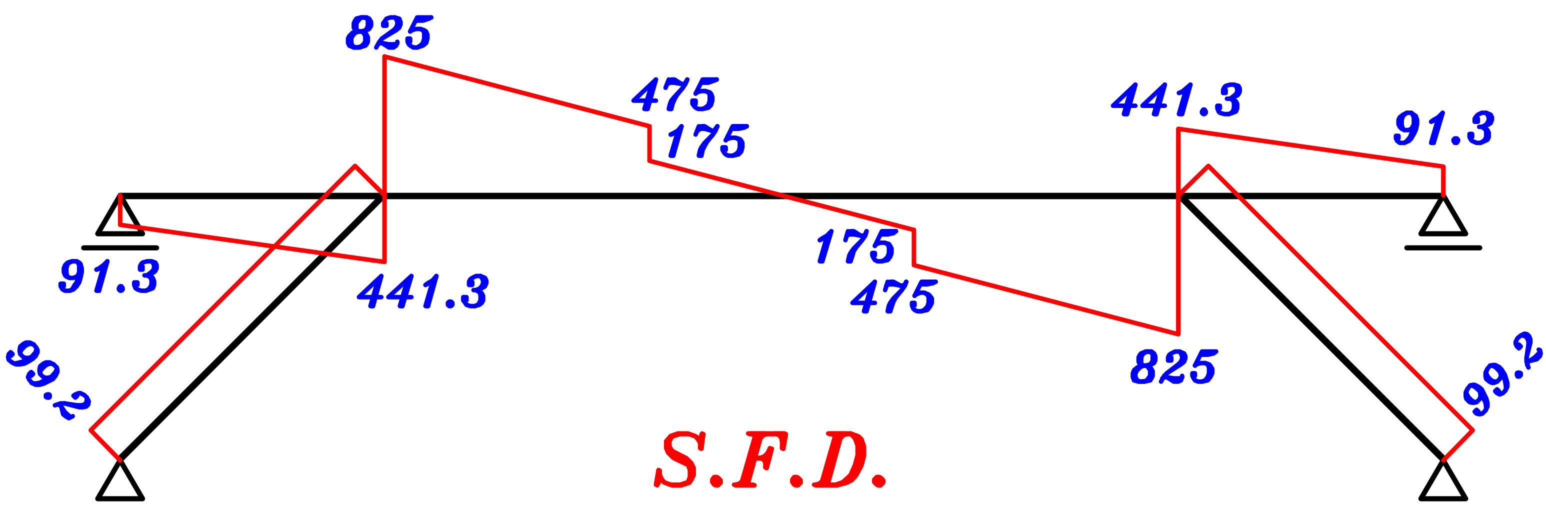
I.F.D. (working)



B.M.D.
(working)

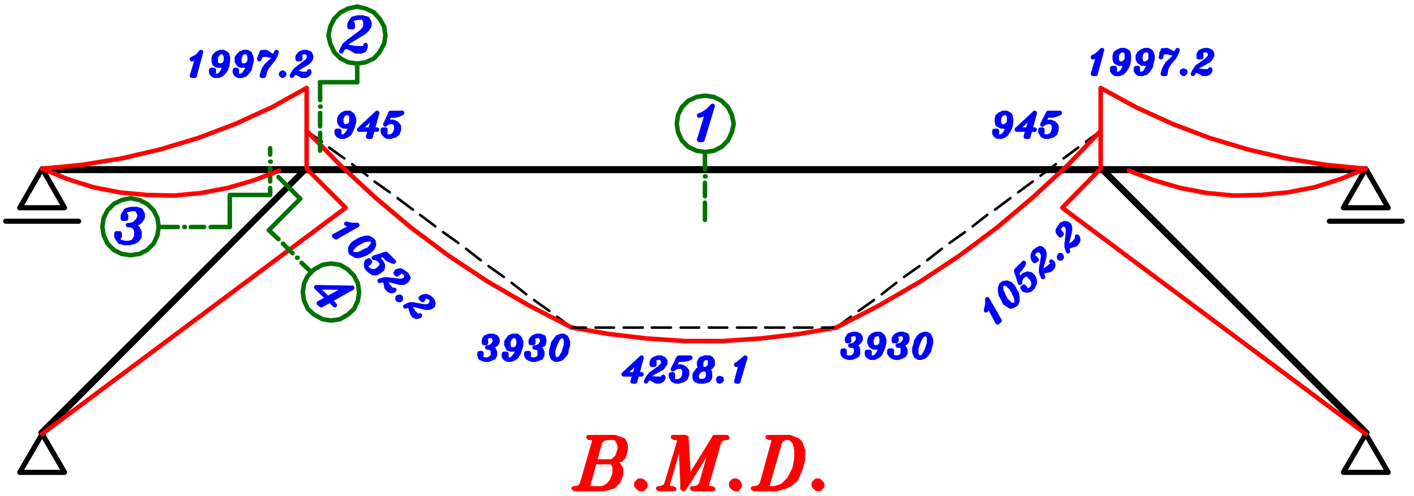


N.F.D.
(working)

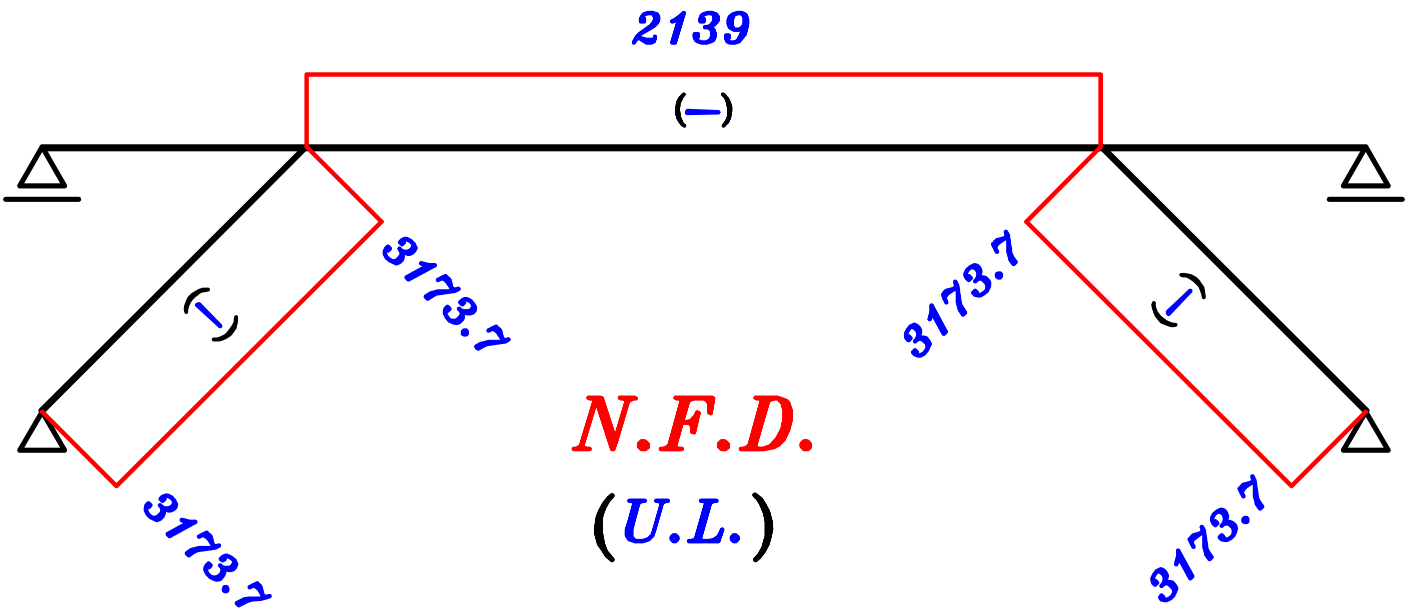


S.F.D.
(working)

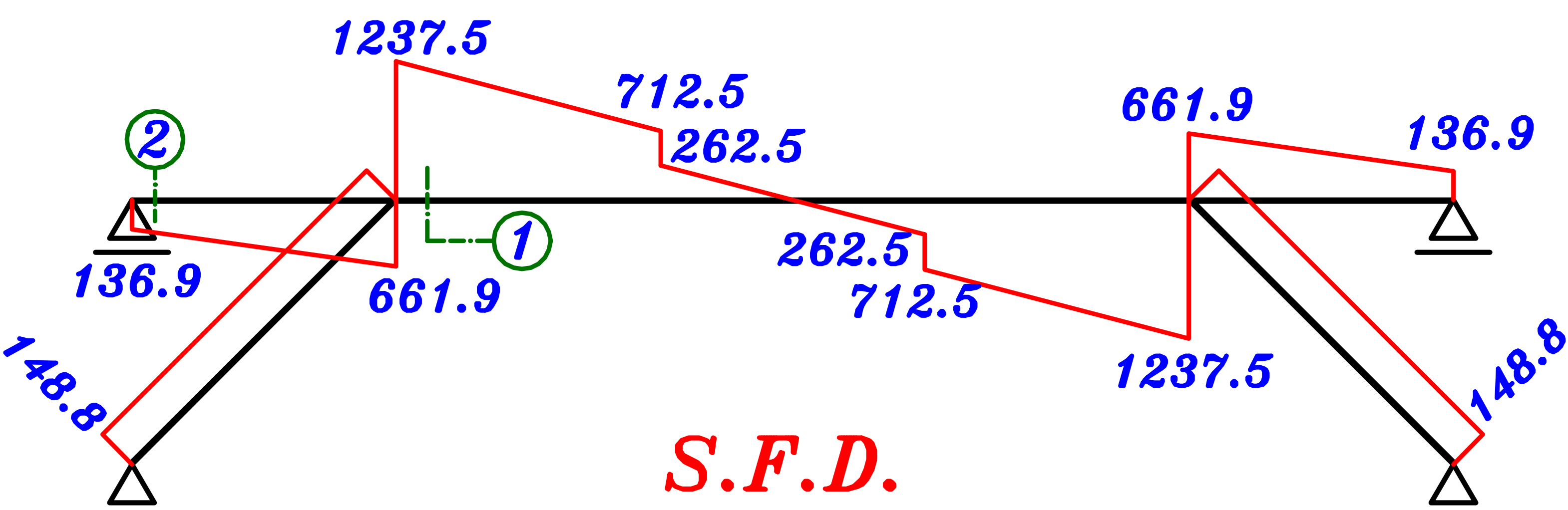
I.F.D. (U.L.)



B.M.D.
(U.L.)



N.F.D.
(U.L.)

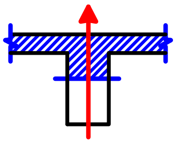


S.F.D.
(U.L.)

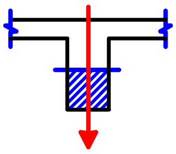
4 - Design the critical sections of the Frame For Bending and shear.

Concrete Dimensions الابعاد التى تم فرضها لل
عاده نستخدمها فى حساب الاوزان فقط و ليس شرط ان نأخذها معنا فى التصميم .

Sec. ① $M_{U.L.} = 4258.1$ kN.m T-Sec.



Sec. ③ $M_{U.L.} = 1997.2$ kN.m R-Sec.



$\because M_T > 2 M_R \quad \therefore$ Design T-Sec. First.

Sec. ① $M = 4258.1$ kN.m , $P = 2139$ kN , $b = 400$ mm

$$d_o = 3.5 \sqrt{\frac{4258.1 * 10^6}{35 * 400}} = 1930.2 \text{ mm} \qquad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)(1930.2) = (2123.2 \rightarrow 2509.2) \text{ mm}$$

Take $\boxed{d = 2200 \text{ mm}}$, $\boxed{t = 2200 + 100 = 2300 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{2139 * 10^3}{35 * 400 * 2300} = 0.066 > 0.04 \quad \therefore$ (Don,t neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{4258.1}{2139} = 2.0 \text{ m} \qquad \therefore \frac{e}{t} = \frac{2.0}{2.3} = 0.86 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - C = 2.0 + \frac{2.30}{2} - 0.10 = 3.05 \text{ m}$$

$$M_s = P * e_s = 2139 * 3.05 = 6523.9 \text{ kN.m}$$

$$\therefore 2200 = C_1 \sqrt{\frac{6523.9 * 10^6}{35 * 400}} \longrightarrow C_1 = 3.22 \longrightarrow J = 0.762$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$\therefore \quad A_s = \frac{6523.9 * 10^6}{0.762 * 360 * 2200} - \frac{2139 * 10^3}{(360 \backslash 1.15)} = 3977.1 \ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 3977.1 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{360}\right) 400 * 2200 = 3253.8 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3977.1 \ mm^2$ $\quad$ 9 ϕ25

$$\therefore \ n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

Sec. 2 $\quad M = 945$ kN.m , $P = 2139$ kN , $b = 400$ mm

$d = 2200$ mm (the same depth of sec. 1)

Check $\quad \dfrac{P}{F_{cu}\, b\, t} = \dfrac{2139 * 10^3}{35 * 400 * 2300} = 0.066 > 0.04 \quad \therefore$ (Don,t neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$e = \dfrac{M}{P} = \dfrac{945}{2139} = 0.44$ m $\quad \therefore \dfrac{e}{t} = \dfrac{0.44}{2.3} = 0.19 < 0.5 \xrightarrow{\text{Use}}$ I.D.

$\therefore$ Use Interaction Diagram

$\zeta = \dfrac{2300 - 200}{2300} = 0.90 \xrightarrow{\text{use}}$ ECCS Design Aids Page 4-23

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{2139 * 10^3}{35 * 400 * 2300} = 0.066 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{945 * 10^6}{35 * 400 * 2300^2} = 0.0127 \end{aligned}\right\} \rho < 1.0 \xrightarrow{\text{Take}} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 35 * 10^{-4} = 3.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 3.5 * 10^{-3} * 400 * 2300 = 3220 \ mm^2$$

$-$ Check $A_{s_{min.}} = \dfrac{0.8}{100} * b * t = \dfrac{0.8}{100} * 400 * 2300 = 7360 \ mm^2$

$A_{s_{Total}} = A_s + A_s` = 2 * 3220 = 6440 \ mm^2 \quad \therefore A_{s_{Total}} < A_{s_{min.}}$

$\therefore$ take $A_s = A_s` = \dfrac{A_{s_{min.}}}{2} = \dfrac{7360}{2} = 3680 \ mm^2$ $\quad$ 8 ϕ25

Sec. ③ $M = 1997.2$ kN.m , $b = 400$ mm

$d = 2200$ mm (the same depth of Sec. ①)

$$\therefore 2200 = C_1 \sqrt{\frac{1997.2 * 10^6}{35 * 400}} \longrightarrow C_1 = 5.82 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1997.2 * 10^6}{0.826 * 360 * 2200} = 3052.9 \ mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 3052.9 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{360}\right) 400 * 2200 = 3253.8 \ mm^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{Use} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \frac{\sqrt{35}}{360}\right) 400 * 2200 = 3253.8 \ mm^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 3052.9 = 3968.8 \ mm^2 \end{array}\right\} \text{الأقل} = 3253.8 \ mm^2 \quad 7 \phi 25$$

Sec. ④ R-Sec. $M = 1052.2$ kN.m , $P = 3173.7$ kN

$$d_o = 3.5 \sqrt{\frac{1052.2 * 10^6}{35 * 400}} = 959.5 \ mm \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(959.5) = (1055.4 \rightarrow 1247.3) \ mm$$

$$\therefore \text{Take} \ \boxed{d = 1200 \ mm} \ , \ \boxed{t = 1200 + 100 = 1300 \ mm}$$

$$\because \boxed{t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)} = 2300 \ mm}$$

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{3173.7 * 10^3}{35 * 400 * 2300} = 0.098 > 0.04 \quad \therefore (\text{Don't neglect } P)$$

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{1052.2}{3173.7} = 0.33\ m \quad \therefore \frac{e}{t} = \frac{0.33}{2.3} = 0.14 < 0.5 \xrightarrow{\text{Use}} \text{I.D.}$$

$\therefore$ Use Interaction Diagram

$$\zeta = \frac{2300 - 200}{2300} = 0.90 \xrightarrow{\text{use}} \text{ECCS Design Aids Page } 4\text{–}23$$

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{3173.7 * 10^3}{35 * 400 * 2300} = 0.098 \\[2ex] \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{1052.2 * 10^6}{35 * 400 * 2300^2} = 0.0142 \end{array}\right\} \rho < 1.0 \xrightarrow{\text{Take}} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 35 * 10^{-4} = 3.5 * 10^{-3}$$

$$A_s = A_s' = \mu * b * t = 3.5 * 10^{-3} * 400 * 2300 = 3220\ mm^2$$

– Check $A_{s\,min.} = \dfrac{0.8}{100} * b * t = \dfrac{0.8}{100} * 400 * 2300 = 7360\ mm^2$

$$A_{s\,Total} = A_s + A_s' = 2 * 3220 = 6440\ mm^2 \quad \therefore A_{s\,Total} < A_{s\,min.}$$

$$\therefore \text{take } A_s = A_s' = \frac{A_{s\,min.}}{2} = \frac{7360}{2} = 3680\ mm^2 \quad 8\phi 25$$

Check Shear.

– Allowable shear stress.

$$- \ q_{cu} = 0.24 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\frac{35}{1.5}} = 1.15\ N\backslash mm^2$$

$$- \ q_{max.} = 0.7 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7 \sqrt{\frac{35}{1.5}} = 3.38\ N\backslash mm^2$$

Sec. 1 $\quad Q = 1237.5\ kN$

$$\therefore \text{Actual shear stress.} = q_U = \frac{Q}{b\, d} = \frac{1237.5 * 10^3}{400 * 2200} = 1.40\ N\backslash mm^2$$

$\therefore\ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8 \backslash m'$

$\therefore$ Use $q_s = q_u - \dfrac{q_{cu}}{2} = \dfrac{n\,A_s (F_y \backslash \delta_s)}{b\ \ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$1.40 - \dfrac{1.15}{2} = \dfrac{2 * 50.3\,(240 \backslash 1.15)}{400 * S} \longrightarrow S = 63.6\ mm < 100\ mm$

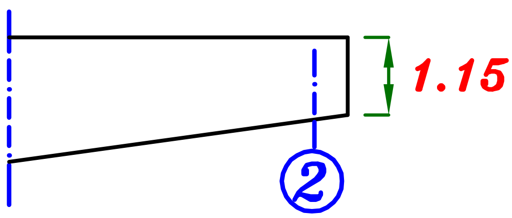
* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$1.40 - \dfrac{1.15}{2} = \dfrac{2 * 78.5\,(240 \backslash 1.15)}{400 * S} \longrightarrow S = 100\ mm$ $\therefore$ o.k.

$\therefore$ No. of stirrups\m' $= \dfrac{1000}{S} = \dfrac{1000}{100} = 10$

$\therefore$ Use Stirrups $\boxed{10\phi 10 \backslash m'}$ 2 branches

Sec. 3 $Q = 136.9\ kN$

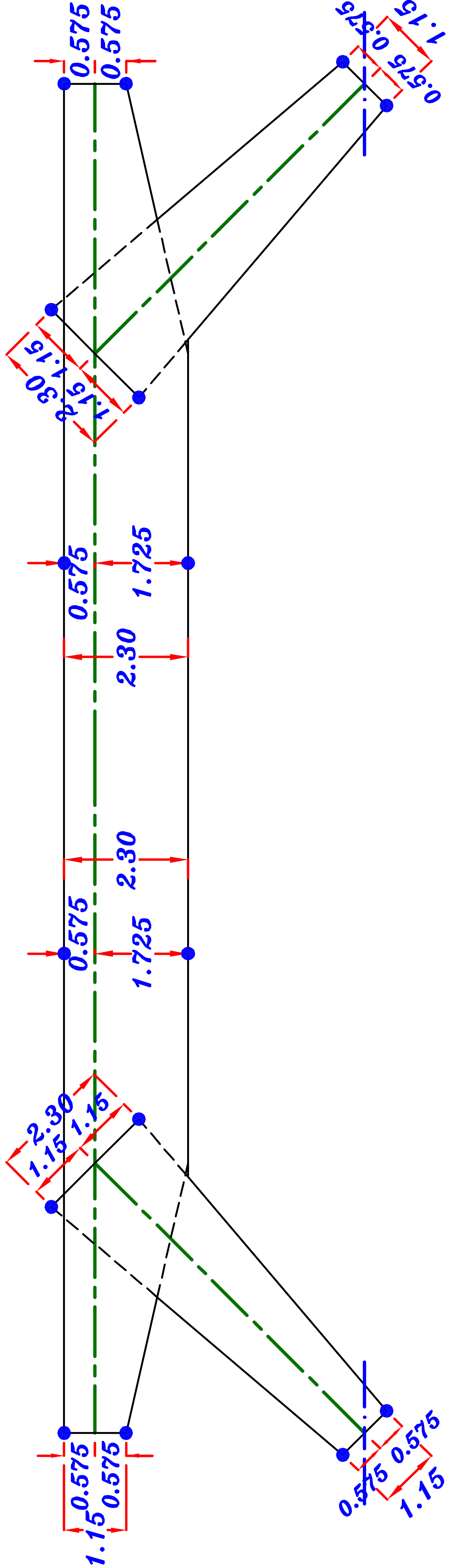


$\therefore$ Actual shear stress. =

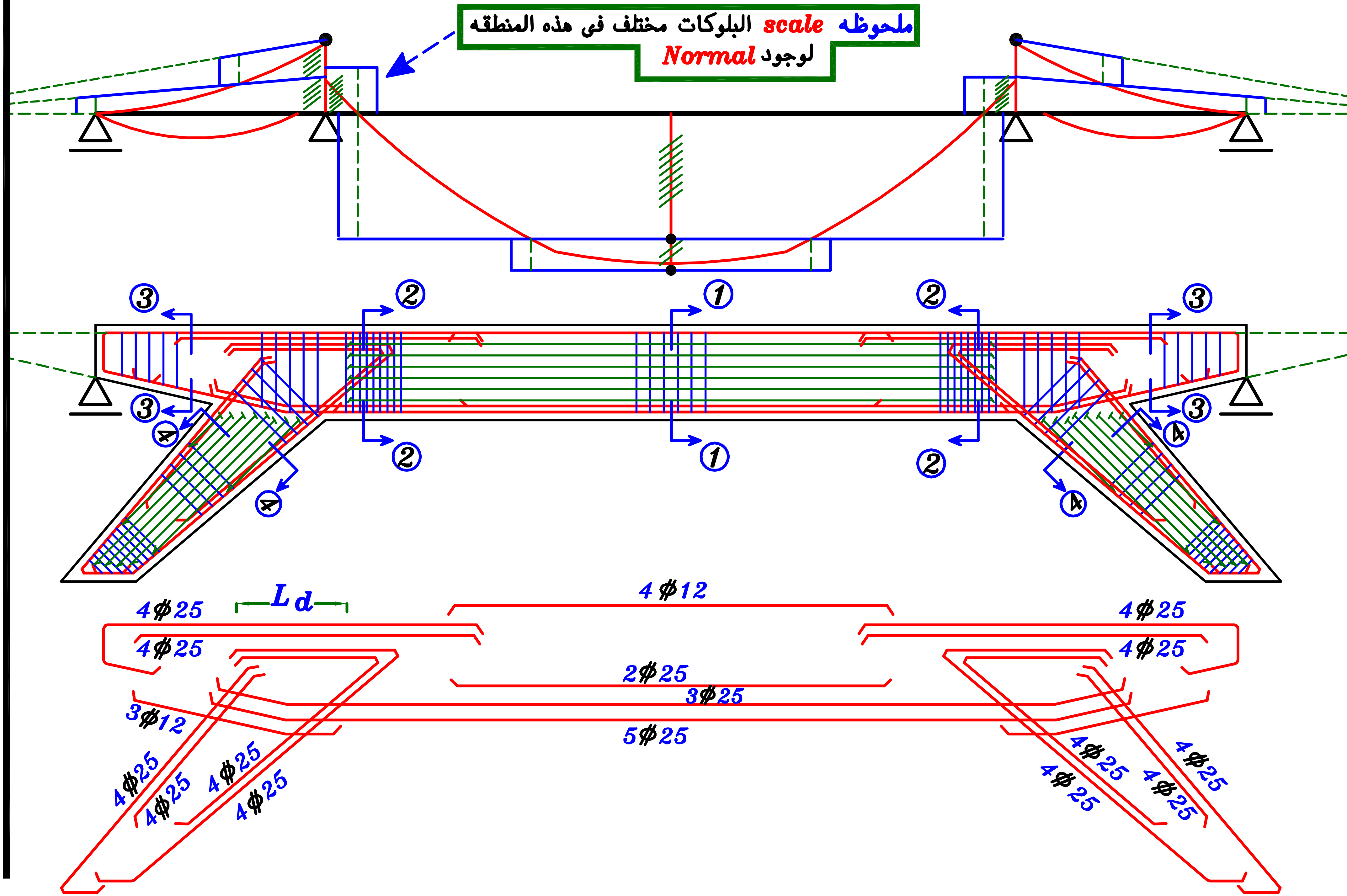
$$q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\ d^2}$$

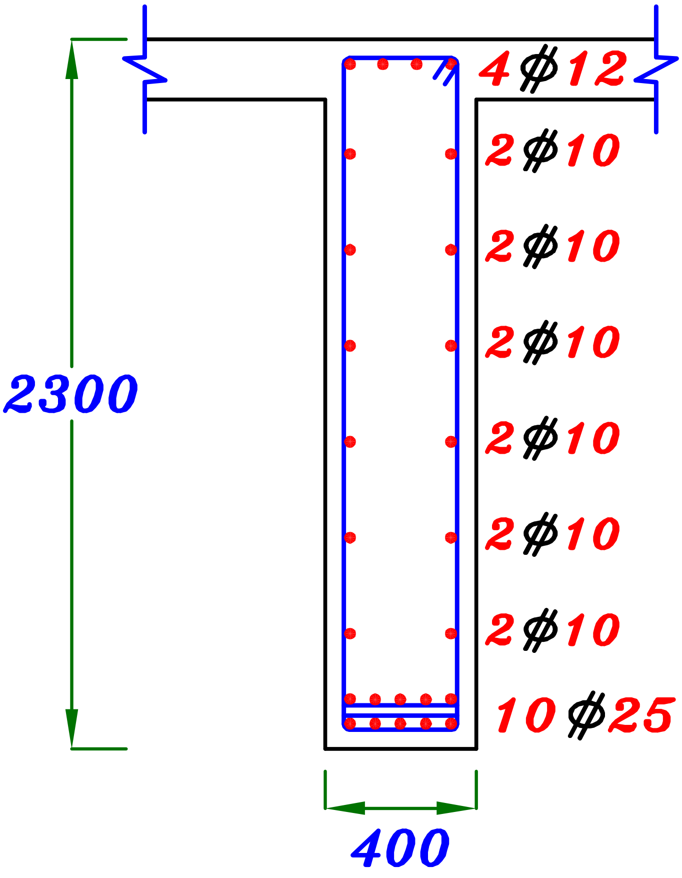
$$= \frac{136.9 * 10^3}{400 * 1050} - ZERO = 0.32\ \ N \backslash mm^2$$

$\therefore\ q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m'}$

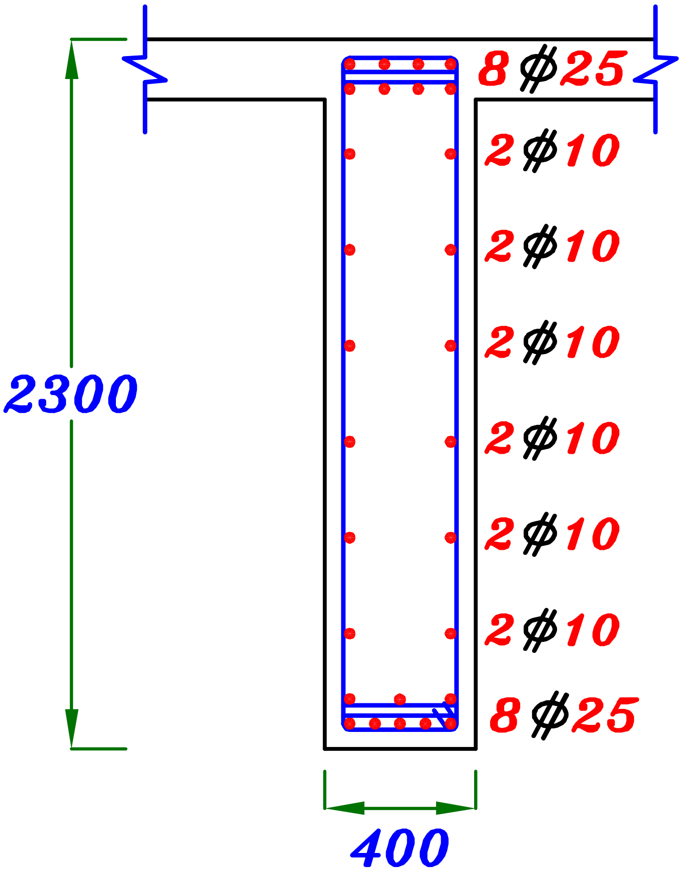

0.575
0.575
1.15
0.575
1.15
0.575
1.15
0.575
0.575
1.725
2.30
2.30
0.575
1.725
2.30
1.15
1.15
0.575
0.575
1.15
0.575
0.575
1.15

R.F.T. of the Frame.

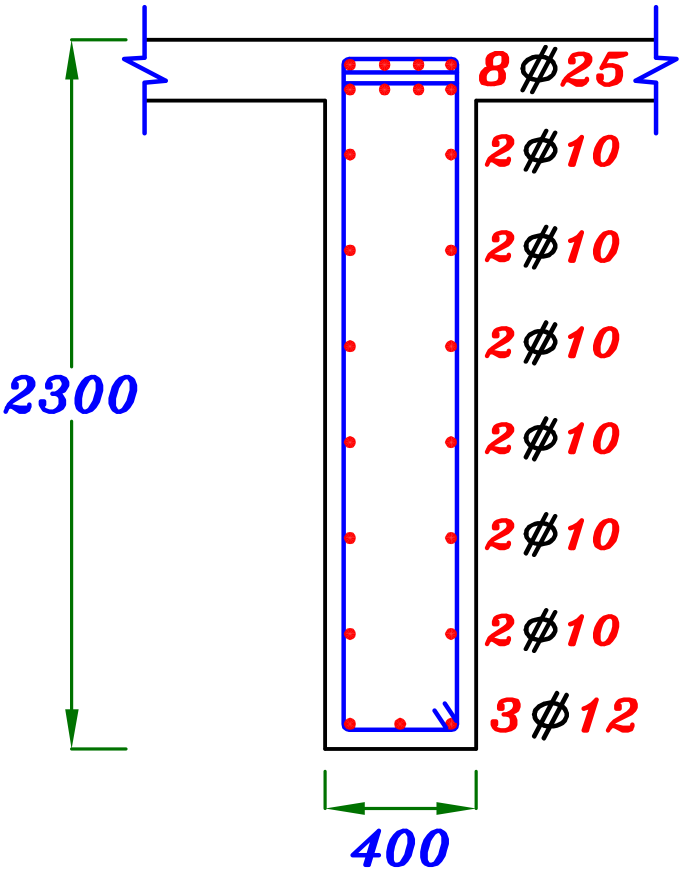




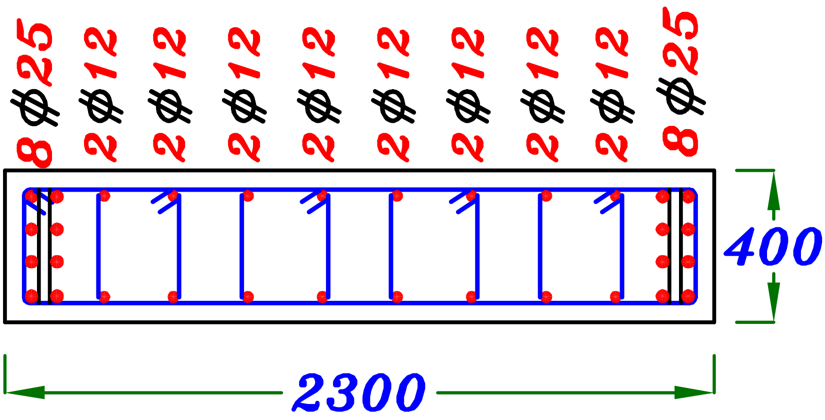
Sec. (1–1)



Sec. (2–2)



Sec. (3–3)



Sec. (4–4)

Example.

Question (1)

Fig. (1) shows a sectional elevation, statical system and load diagram For a reinforced concrete stadium ring Frame. The ring stadium is covered by reinforced concrete slabs supported by a system of secondary beams and Frames (F), spaced at 6.0 m.

For an intermediate panel, It is required to :

1- Without any calculation but with reasonably assumed concrete dimensions, draw sectional For the ring Frame, Showing the dimensions of all concrete elements. to scale 1:50

2- Draw B.M.D., N.F.D. & S.F.D. For case of total load only of an intermediate Frame (F) Using the given Ultimate limit loads.

3- Design the critical sections For the intermediate Frame (F), to satisfy both bending moment and normal Forces.

4- Design the critical sections For the intermediate Frame (F), to satisfy the shearing Force.

5- Using the moment of resistance principle. draw the details of reinforcement For the Frame in elevation (scale 1:50) and cross sections (scale 1:20)

Data:

$t_s = 140$ mm , Spacing = 6.0 m

$b_{(beam)} = 250$ mm , $b_{(Frame)} = 400$ mm

Concrete Grade. C 30 $F_{cu} = 30$ MPa

Steel Grade: st. 52 $F_y = 360$ MPa

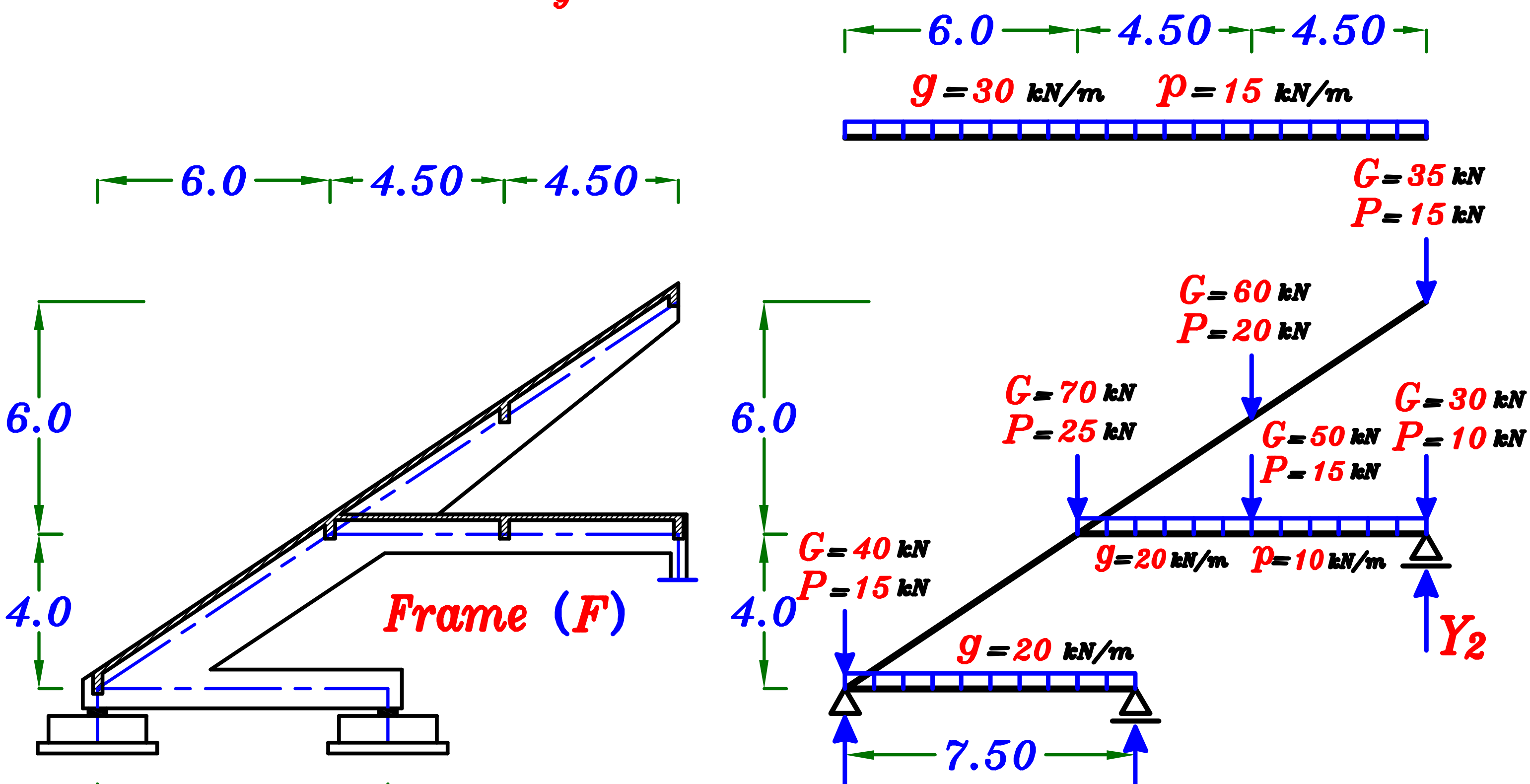
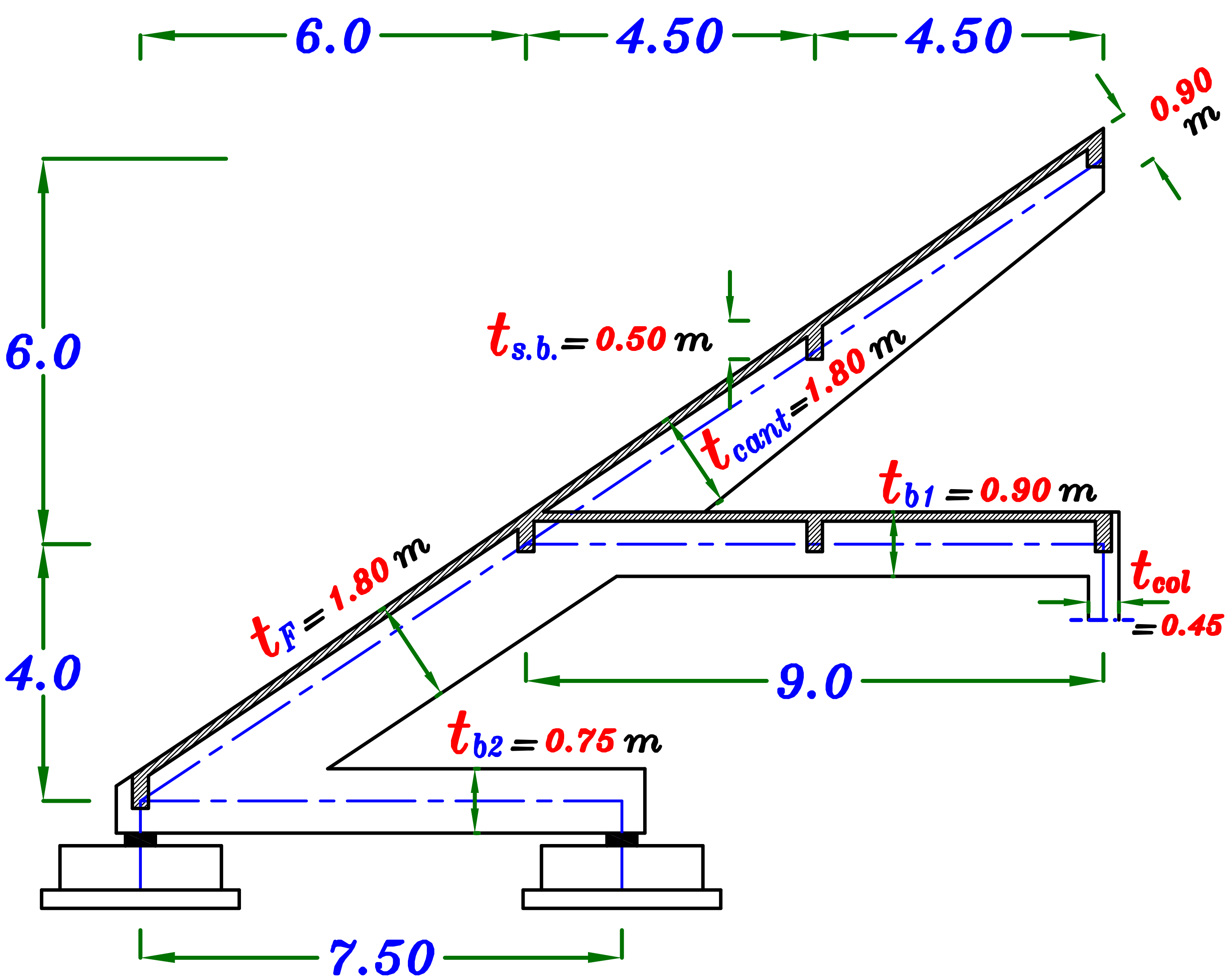


Figure (1)

Question (1)

1- Without any calculation but with reasonably assumed concrete dimensions, draw sectional For the ring Frame, Showing the dimensions of all concrete elements. to scale 1:50



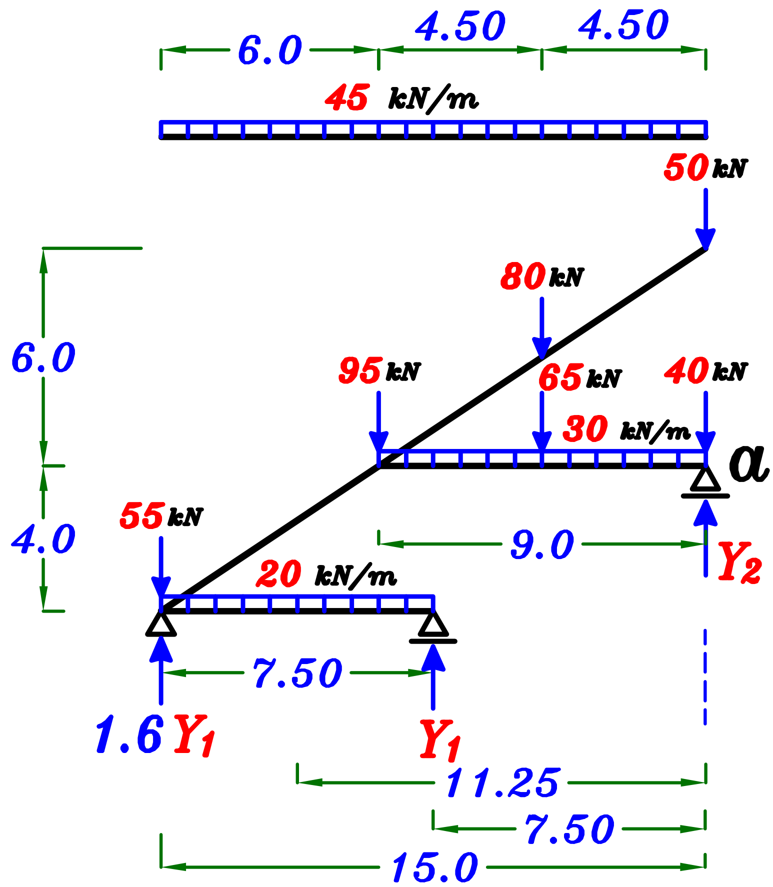
$$t_{b1} = \frac{L}{10} = \frac{9.0}{10} = 0.90\ m \quad , \quad t_{b2} = \frac{L}{10} = \frac{7.5}{10} = 0.75\ m$$

$$t_{cant.} = \frac{L}{5} = \frac{9.0}{5} = 1.80\ m \quad , \quad t_F \simeq t_{cant.} \simeq 1.80\ m$$

$$t_{col.} = \frac{t_{b1}}{2} = \frac{0.9}{2} = 0.45\ m$$

$$t_{s.b.} = \frac{spacing}{12} = \frac{6.0}{12} = 0.50\ m$$

2- Draw B.M.D., N.F.D. & S.F.D. For case of total load only of an intermediate Frame (F) Using the given Ultimate limit loads.

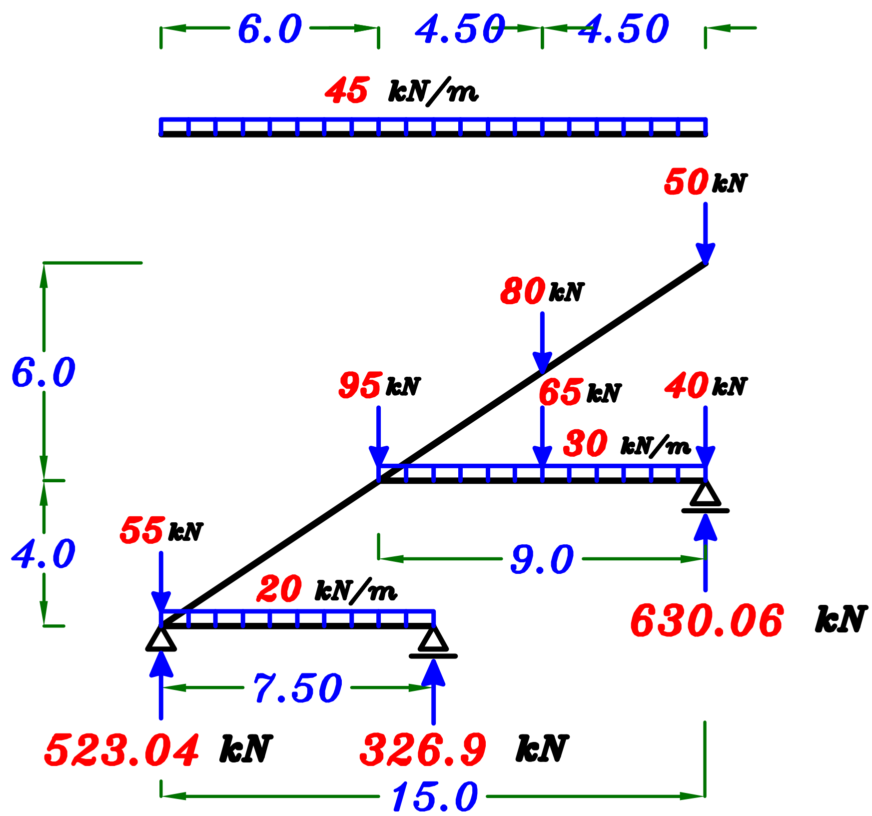


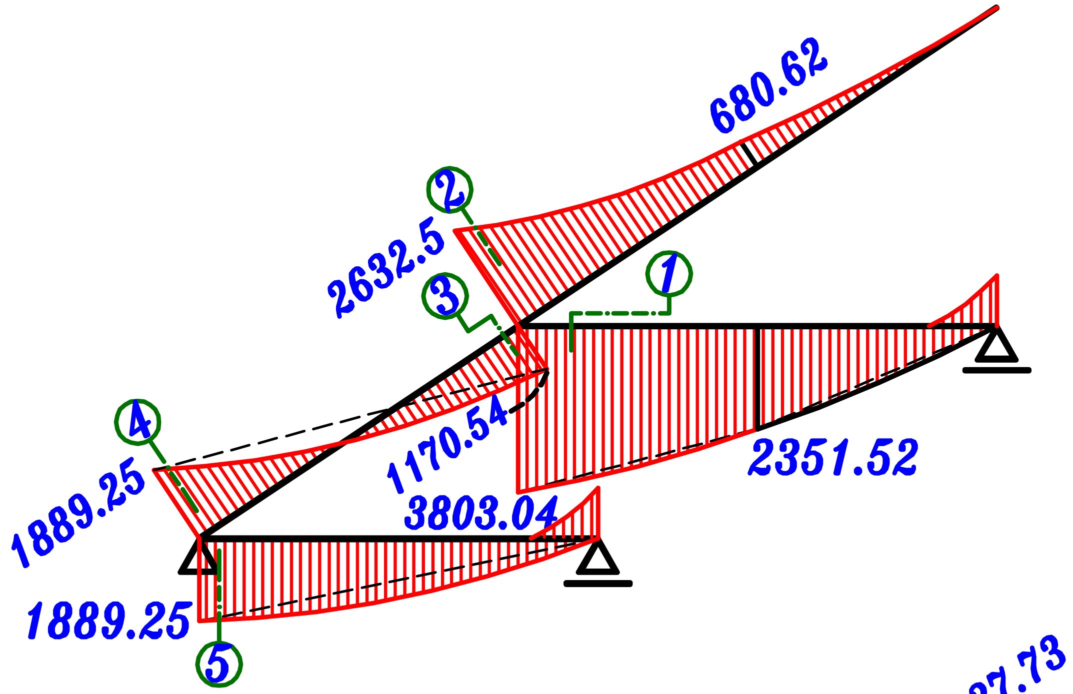
$$\sum M_a = Zero$$

$$45(15)(7.5) + 30(9)(4.5) + 80(4.5) + 65(4.5) + 95(9) + 20(7.5)(11.25)$$

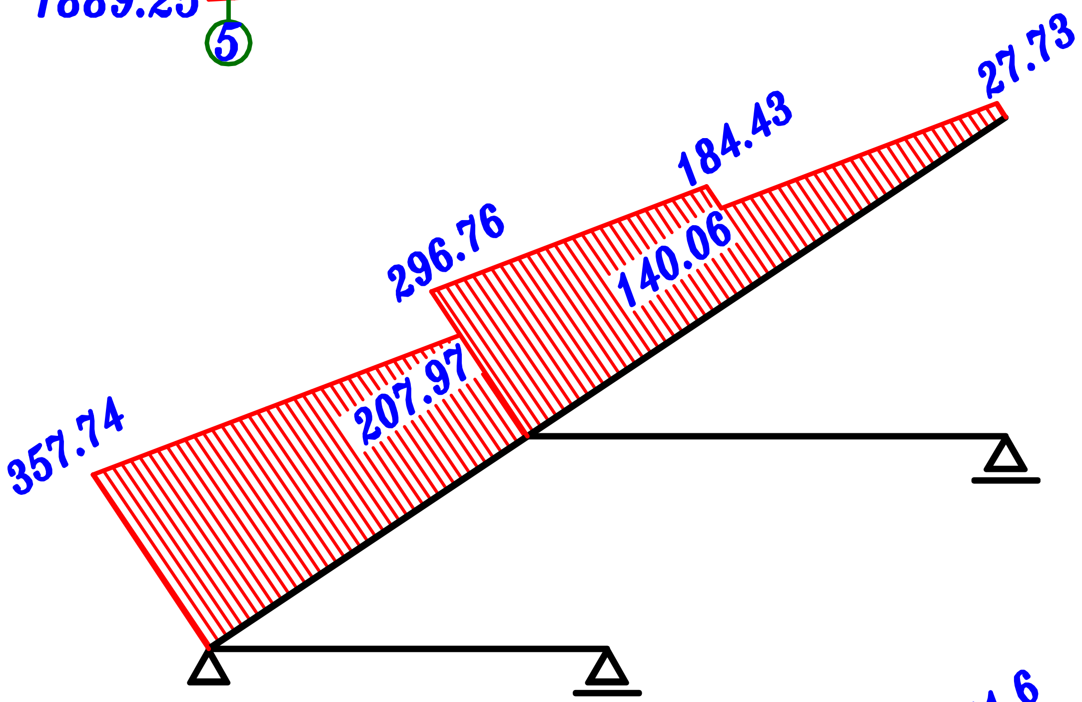
$$+55(15) - (1.6Y_1)(15) - (Y_1)(7.5) = Zero \qquad \boxed{Y_1 = 326.9 \text{ kN}}$$

$$\sum Y = Zero \qquad \boxed{Y_2 = 630.06 \text{ kN}}$$

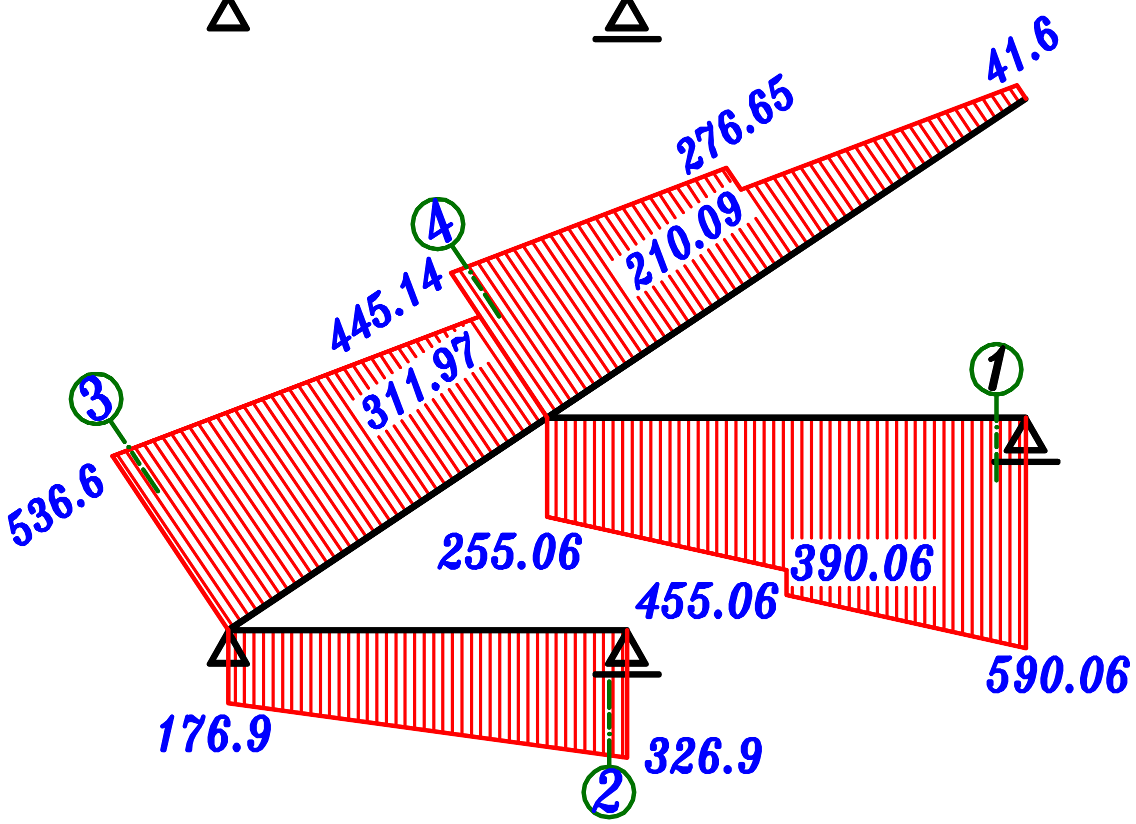




B.M.D. (U.L.)



N.F.D. (U.L.)

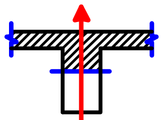


S.F.D. (U.L.)

3–Design the critical sections For the intermediate Frame (F), to satisfy both bending moment and normal Forces.

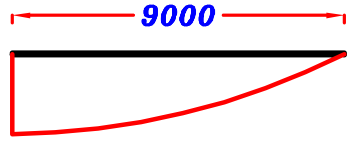
الابعاد التى تم فرضها لل Concrete Dimensions عاده نستخدمها فى حساب الاوزان فقط و ليس شرط ان نأخذها معنا فى التصميم .

Sec. ① $M = 3803.04\ kN.m$, $P = Zero\ kN$, $b = 400\ mm$



T–Sec.

$$B = \begin{cases} C.L.-C.L. = Spacing = 6.0\,m = 6000\ mm \\ 16\ t_s + b = 16 * 140 + 400 = 2640\ mm \\ K\dfrac{L}{5} + b = 1.0 * \dfrac{9000}{5} + 400 = 2200\ mm \end{cases}$$



$\boxed{B = 2200\ mm}$

Take $C_1 = 6.0 \longrightarrow J = 0.826$

$$d = 6.0\sqrt{\frac{3803.04 * 10^6}{30 * 2200}} = 1440.2\ mm \quad (T\text{–}Sec.)$$

Take $\boxed{d = 1500\ mm}$, $\boxed{t = 1500 + 100 = 1600\ mm}$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{3803.04 * 10^6}{0.826 * 360 * 1440.2} = 8880.2\ mm^2$$

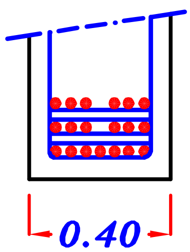
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 8880.2\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1500 = 2054\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d \quad \therefore$ Take $A_s = A_{s_{req.}} = 8880.2\ mm^2$ 19Φ25

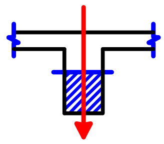
$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

19Φ25



Sec. ②

$M = 2632.5$ kN.m , $P = 296.76$ kN , $b = 400$ mm



R–Sec.

$$d_o = 3.5\sqrt{\frac{2632.5 * 10^6}{30 * 400}} = 1639.3 \text{ mm} \quad (\text{as R–Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)(1639.3) = (1803.2 \rightarrow 2131) \text{ mm}$$

Take $d = 1900$ mm , $t = 1900 + 100 = 2000$ mm

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{296.76 * 10^3}{30 * 400 * 2000} = 0.012 < 0.04 \quad \therefore (\text{Neglect } P)$$

$\therefore$ Take $d = d_o = 1639.3$ mm

$\therefore$ Take $\boxed{d = 1700 \text{ mm}}$, $\boxed{t = 1800 \text{ mm}}$

$\because \quad C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{2632.5 * 10^6}{0.780 * 360 * 1639.3} = 5718.9 \text{ mm}^2$$

Check $A_{s_{min.}}$

$$A_{s_{req.}} = 5718.9 \text{ mm}^2$$

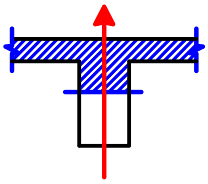
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1700 = 2327.8 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 5718.9 \text{ mm}^2$ $\quad 12\phi25$

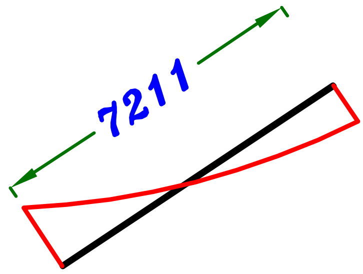
Sec. 3 $M = 1170.54\ kN.m$, $P = 207.97\ kN$, $b = 400\ mm$

$d = 1700\ mm$ (the same depth of Sec. 2)

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{207.97 * 10^3}{30 * 400 * 1800} = 0.0096 < 0.04 \quad \therefore$ (Neglect P)

T–Sec. 

$$B = \begin{cases} C.L. - C.L. = Spacing = 6.0\,m = 6000\ mm \\ 16\ t_s + b = 16 * 140 + 400 = 2640\ mm \\ K\,\dfrac{L}{5} + b = 0.8 * \dfrac{7211}{5} + 400 = 1553.7\ mm \end{cases}$$



$\boxed{B = 1553.7\ mm}$

$$\therefore\ 1700 = C_1 \sqrt{\frac{1170.54 * 10^6}{30 * 1553.7}} \longrightarrow C_1 = 10.7 \longrightarrow J = 0.826$$

$$\therefore\ A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1170.54 * 10^6}{0.826 * 360 * 1700} = 2315.5\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s\,req.} = 2315.5\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1700 = 2327.8\ mm^2$$

$$\therefore\ \mu_{min.}\, b\, d > A_{s\,req.} \xrightarrow{Use} A_{s_{min.}}$$

$$A_{s_{min.}} = 0.225 * \frac{\sqrt{F_{cu}}}{F_y}\, b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1900 = 2327.8$$

$$1.3\ A_{s\,req.} = 1.3 * 2315.5 = 3010.1$$

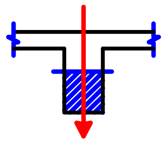
الأقل $= 2327.8$

st. 360/520 $\qquad \dfrac{0.15}{100}\, b\, d = \dfrac{0.15}{100} * 400 * 1900 = 1140$

الأكبر $= 2327.8\ mm^2$

$5 \phi 25$

Sec. ④ $M = 1889.25\ kN.m$, $P = 357.74\ kN$, $b = 400\ mm$



R–Sec.

$d = 1700\ mm$ (the same depth of Sec. ②)

Check $\dfrac{P}{F_{cu}\,b\,t} = \dfrac{357.74 * 10^3}{30 * 400 * 1800} = 0.016 < 0.04$ $\therefore$ (Neglect P)

$$\therefore 1700 = C_1 \sqrt{\frac{1889.25 * 10^6}{30 * 400}} \longrightarrow C_1 = 4.28 \longrightarrow J = 0.813$$

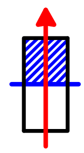
$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{1889.25 * 10^6}{0.813 * 360 * 1700} = 3797.0\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 3797.0\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1700 = 2327.8\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$ $\therefore$ Take $A_s = A_{s_{req.}} = 3797.0\ mm^2$ $\quad$ 8 ϕ25

Sec. ⑤ $M = 1889.25\ kN.m$, $P = Zero\ kN$, $b = 400\ mm$



R–Sec.

Take $C_1 = 3.5 \longrightarrow J = 0.78$

$$d = 3.5 \sqrt{\frac{1889.25 * 10^6}{30 * 400}} = 1388.7\ mm \quad (R\text{–}Sec.)$$

Take $\boxed{d = 1400\ mm}$, $\boxed{t = 1400 + 100 = 1500\ mm}$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{1889.25 * 10^6}{0.780 * 360 * 1388.7} = 4844.9\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 4844.9\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 400 * 1400 = 1917\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$ $\therefore$ Take $A_s = A_{s_{req.}} = 4844.9\ mm^2$ $\quad$ 10 ϕ25

4- Design the critical sections For the intermediate Frame (F) to satisfy the shearing Force.

Check Shear.

- Allowable shear stress.

- $q_{cu} = 0.24 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\frac{30}{1.5}} = 1.07 \ N\backslash mm^2$

- $q_{max.} = 0.7 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7 \sqrt{\frac{30}{1.5}} = 3.13 \ N\backslash mm^2$

Sec. 1 $Q = 590.06 \ kN \quad , \quad d = 1500 \ mm$

$$q_U = \frac{Q}{b\,d} = \frac{590.06 * 10^3}{400 * 1500} = 0.98 \ N\backslash mm^2$$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8\backslash m'}$

Sec. 2 $Q = 326.9 \ kN \quad , \quad d = 1400 \ mm$

$$q_U = \frac{Q}{b\,d} = \frac{326.9 * 10^3}{400 * 1400} = 0.583 \ N\backslash mm^2$$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8\backslash m'}$

Sec. 3 $Q = 536.6 \ kN \quad , \quad d = 1700 \ mm$

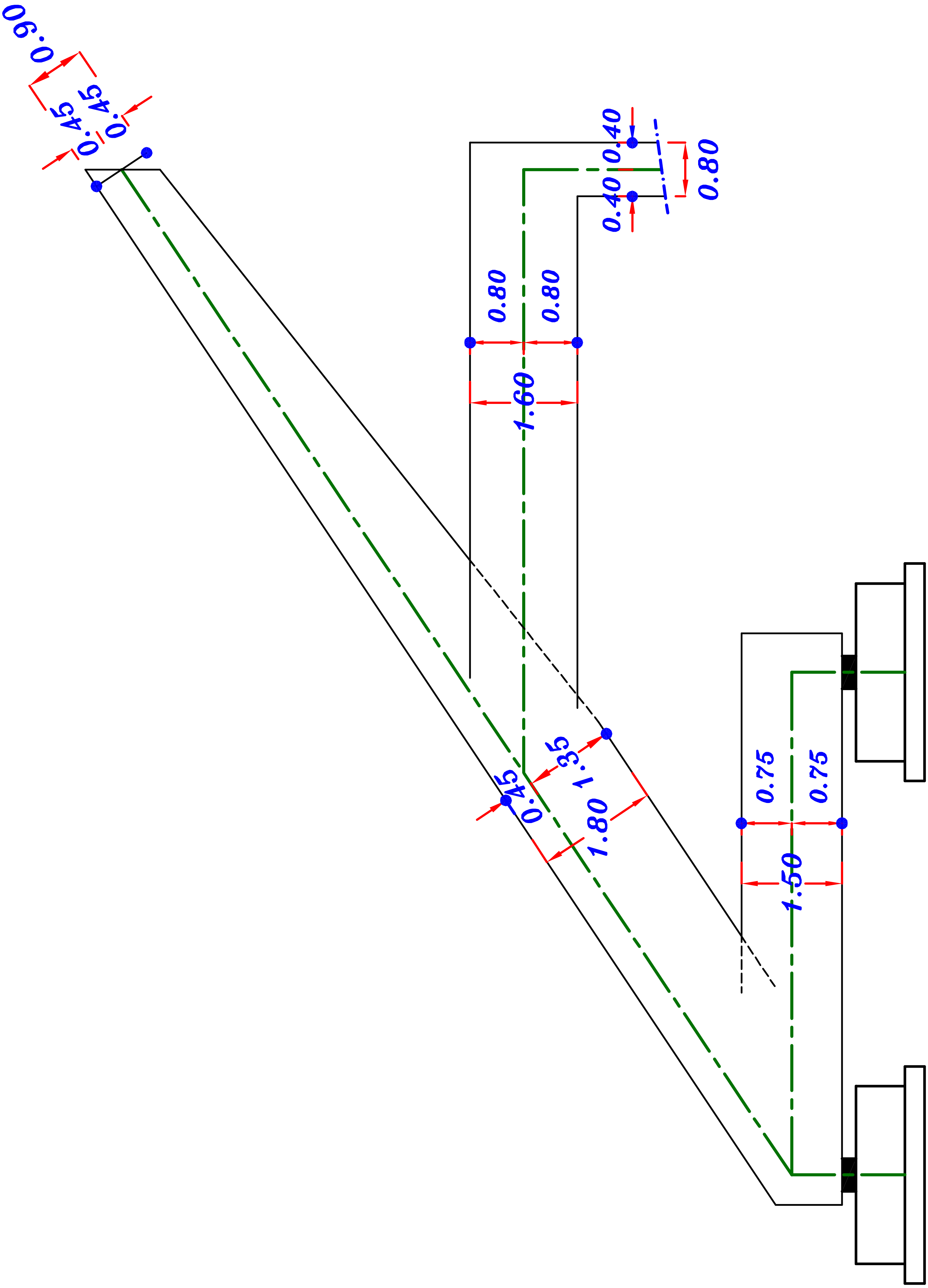
$$q_U = \frac{Q}{b\,d} = \frac{536.6 * 10^3}{400 * 1700} = 0.79 \ N\backslash mm^2$$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8\backslash m'}$

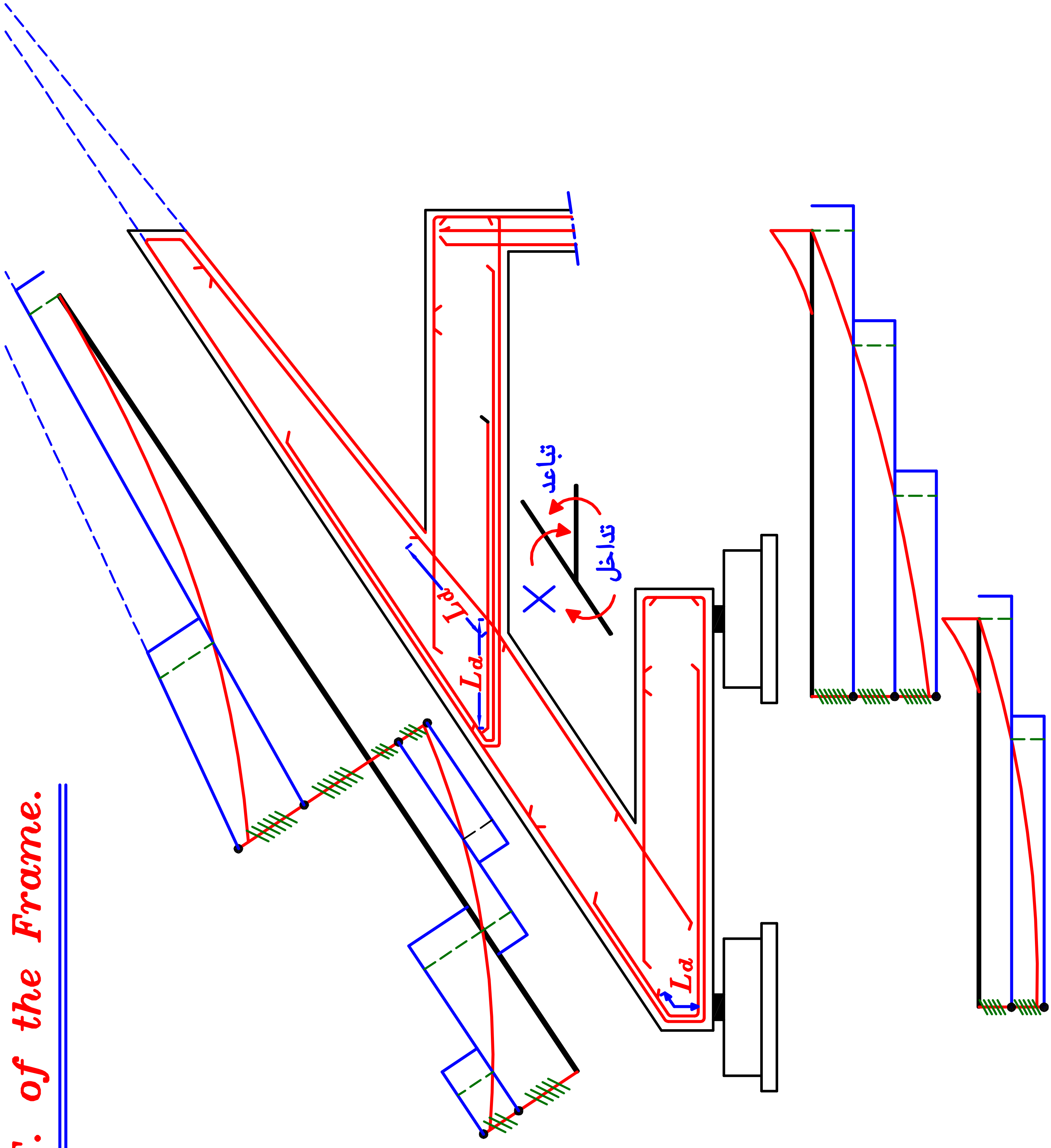
Sec. 4 $Q = 445.14 \ kN \quad , \quad d = 1700 \ mm$

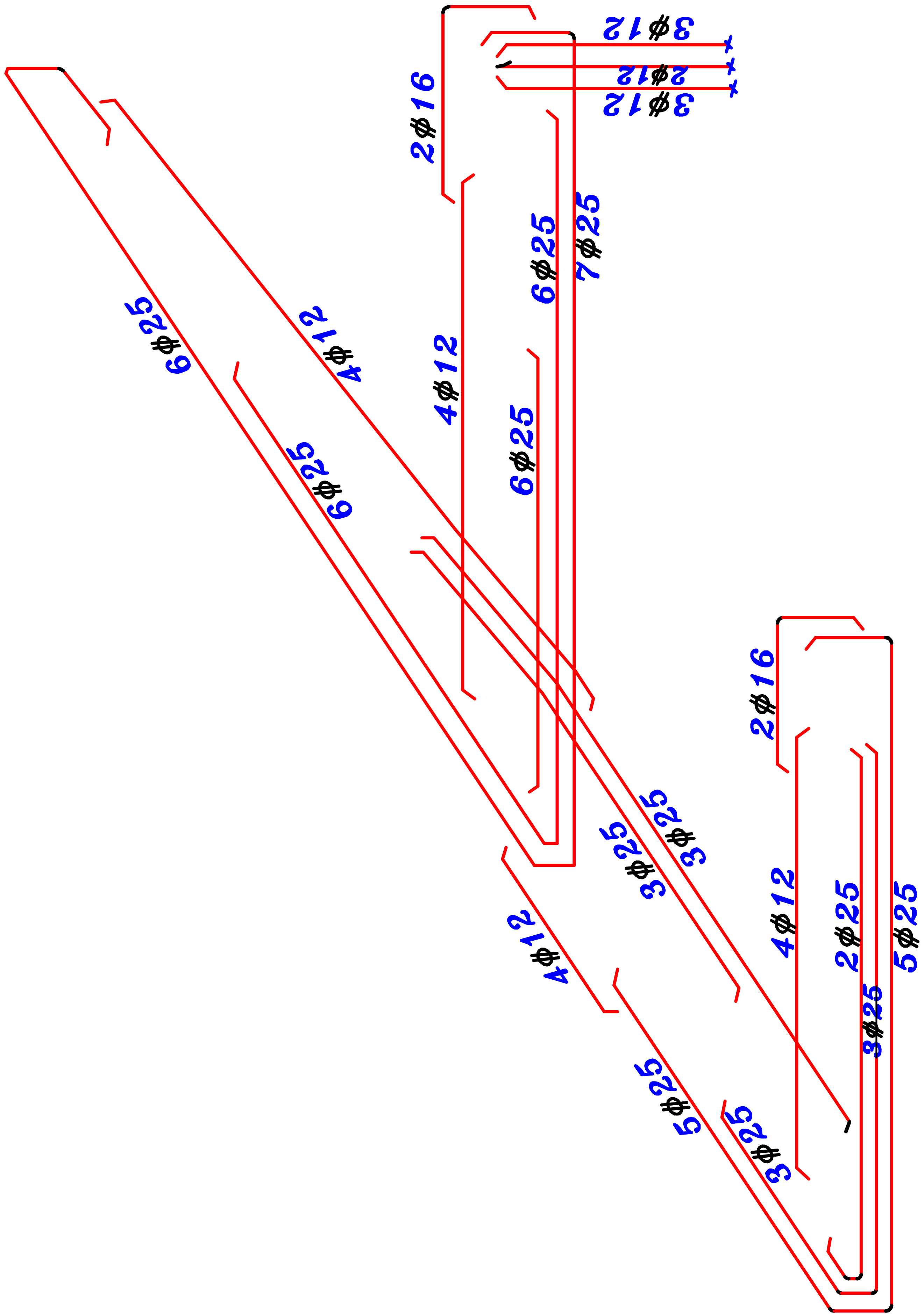
$$q_U = \frac{Q}{b\,d} = \frac{445.14 * 10^3}{400 * 1700} = 0.65 \ N\backslash mm^2$$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8\backslash m'}$

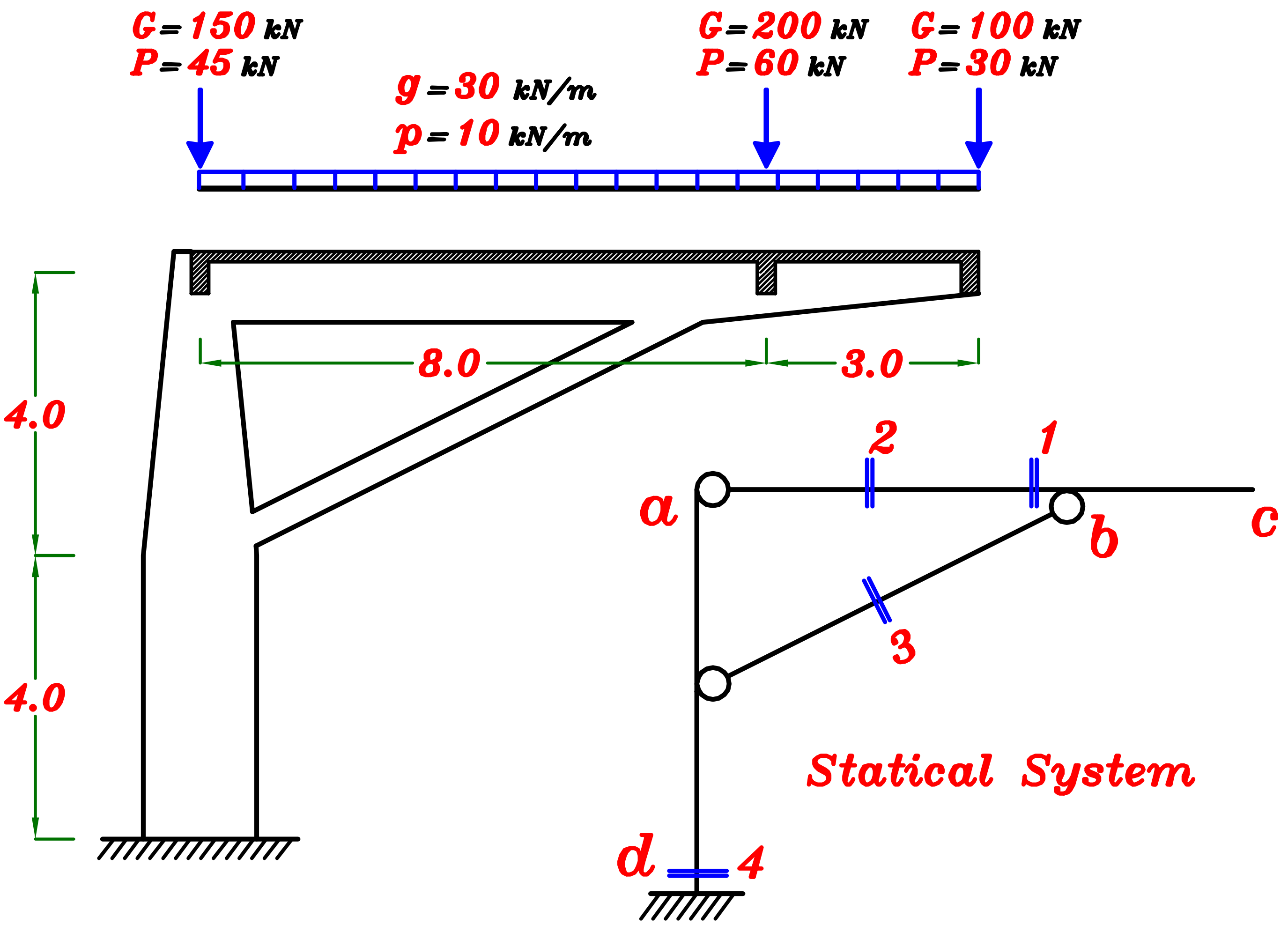

0.90
0.45
0.45
0.40
0.40
0.80
0.80
0.80
1.60
1.35
0.45
1.80
0.75
0.75
1.50

R.F.T. of the Frame.




3Φ12
2Φ12
3Φ12
2Φ16
6Φ25
7Φ25
6Φ25
4Φ12
4Φ12
6Φ25
6Φ25
3Φ25
3Φ25
2Φ16
4Φ12
2Φ25
5Φ25
3Φ25
4Φ12
5Φ25
3Φ25

Example.



The Figure shows the concrete dimensions and statical system of a repeated Frame used as a car shed. The spacing between Frames is **6.0 m** and each Frame carries a group of secondary beams as shown in the Figure. **It is required to:**

1- Draw the **S.F.D., N.F.D. & B.M.D.** For one intermediate Frame.

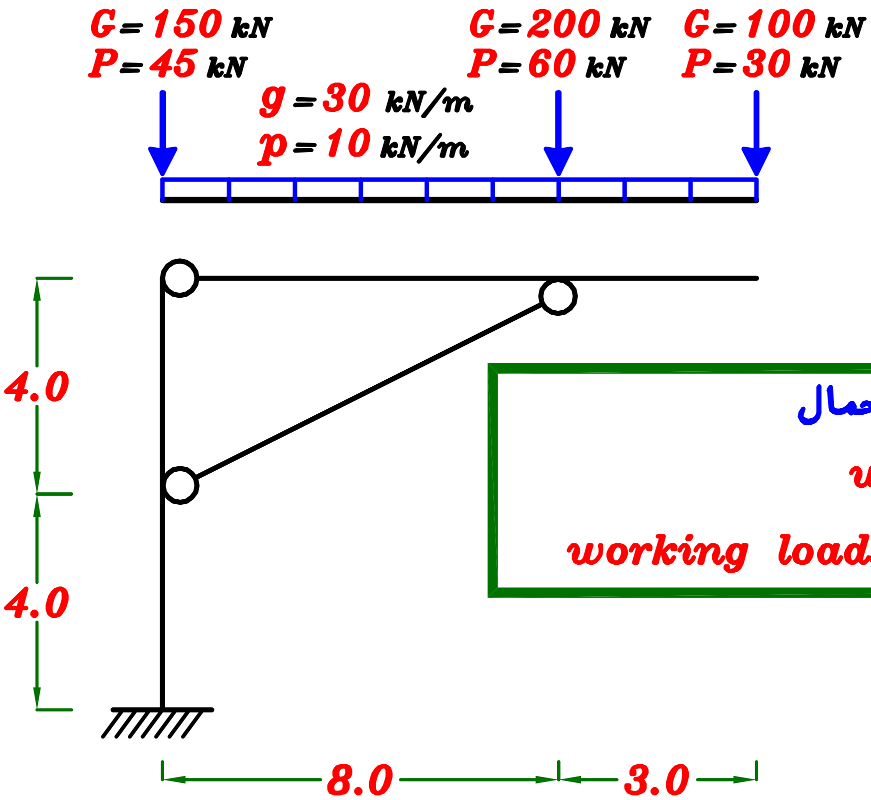
2- Assuming that the column **(a d)** is an unbraced column, calculate the bending moments on the column **in & out** of plane due to buckling.

3- Design the critical sections **(1 → 4)** of the Frame.

4- Draw the shear stress diagram of members **a-b-c** and design its critical sections For shear.

5- Draw the details of the reinforcement of the Frame using the moment of resistance diagram For the bars curtailment For members **a-b-c** and **a-d** in elevation to scale **1:50** and in sections to scale **1:20**

- Use $F_{cu} = 35$ MPa
- Steel used is St. 400/600
- $b = 450$ mm (**Frame**)
- Slabs thickness = 150 mm
- Floor Cover = 2.0 kN/m^2
- Live Load = 1.0 kN/m^2



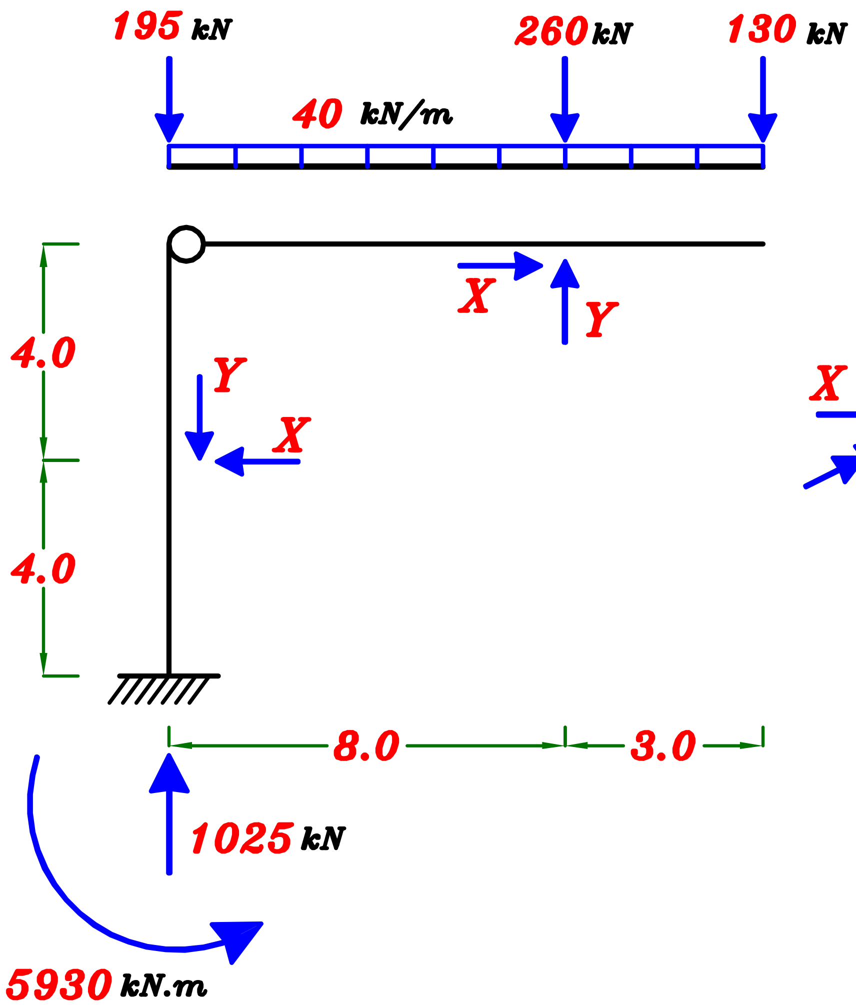
بما انه لم يتم ذكر اذا ما كانت الاحمال

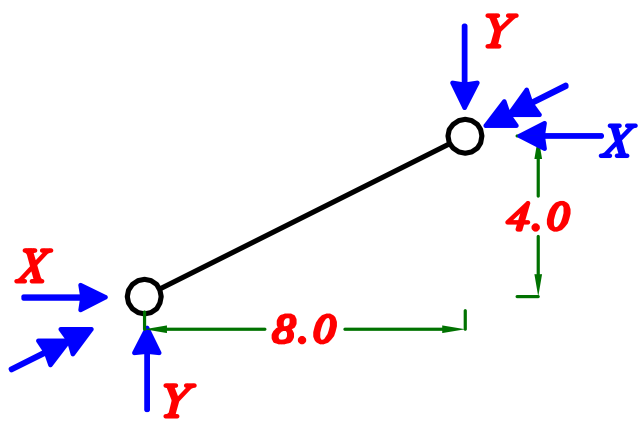
working or ultimate limit

اذا من الطبيعي ان تكون الاحمال *working loads*

1 – Draw the S.F.D. , N.F.D. & B.M.D. For one intermediate Frame.

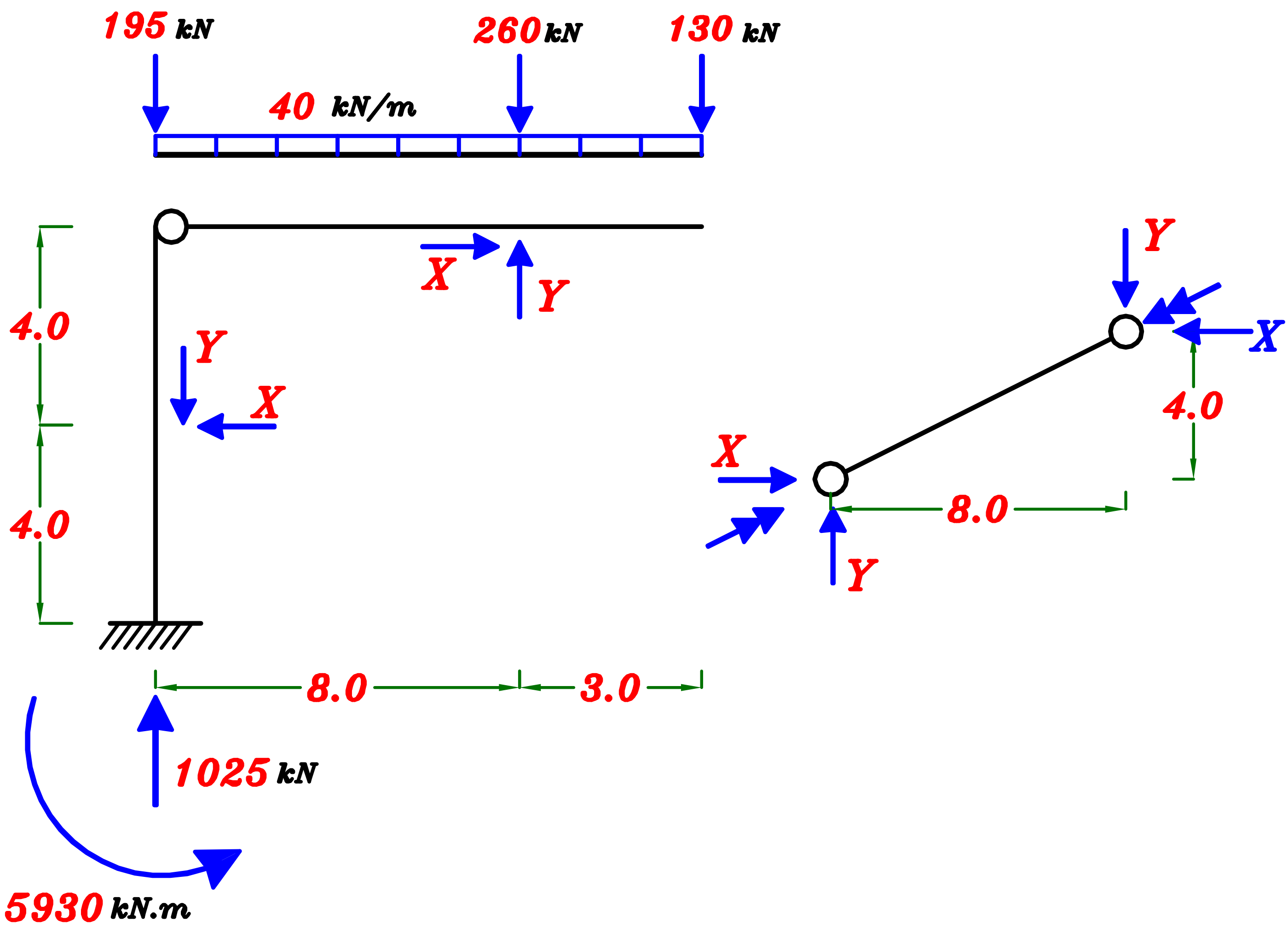
بما انه لم يتم طلب عمل حالات تحميل على الـ *Frame* اذا سنأخذ كل الاحمال *Total*

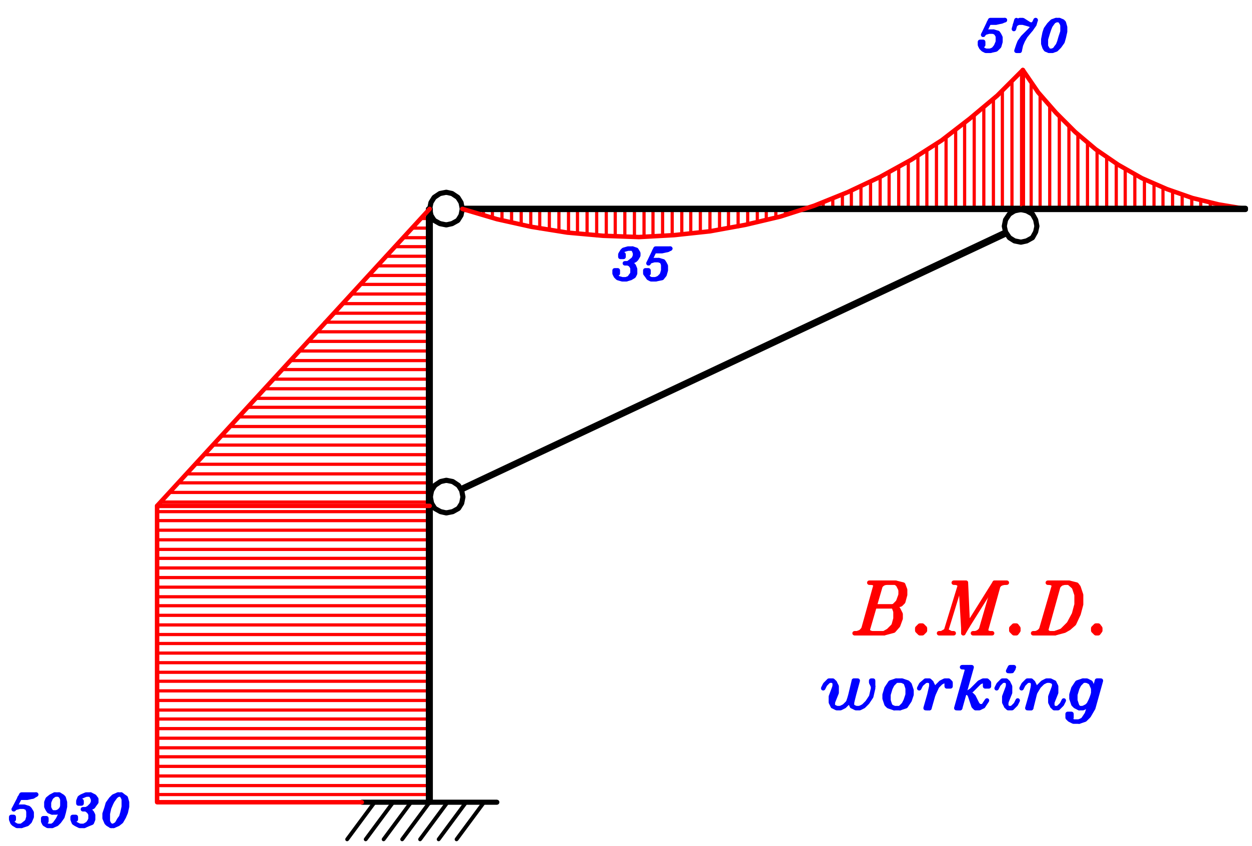


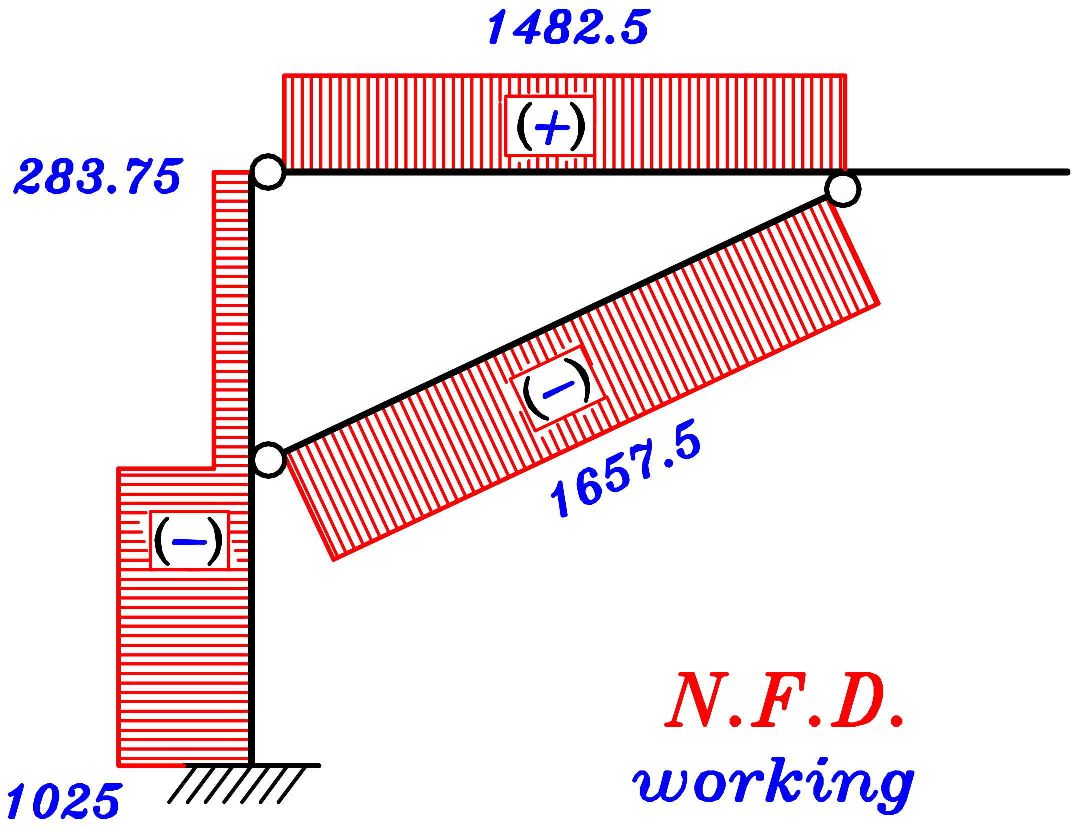


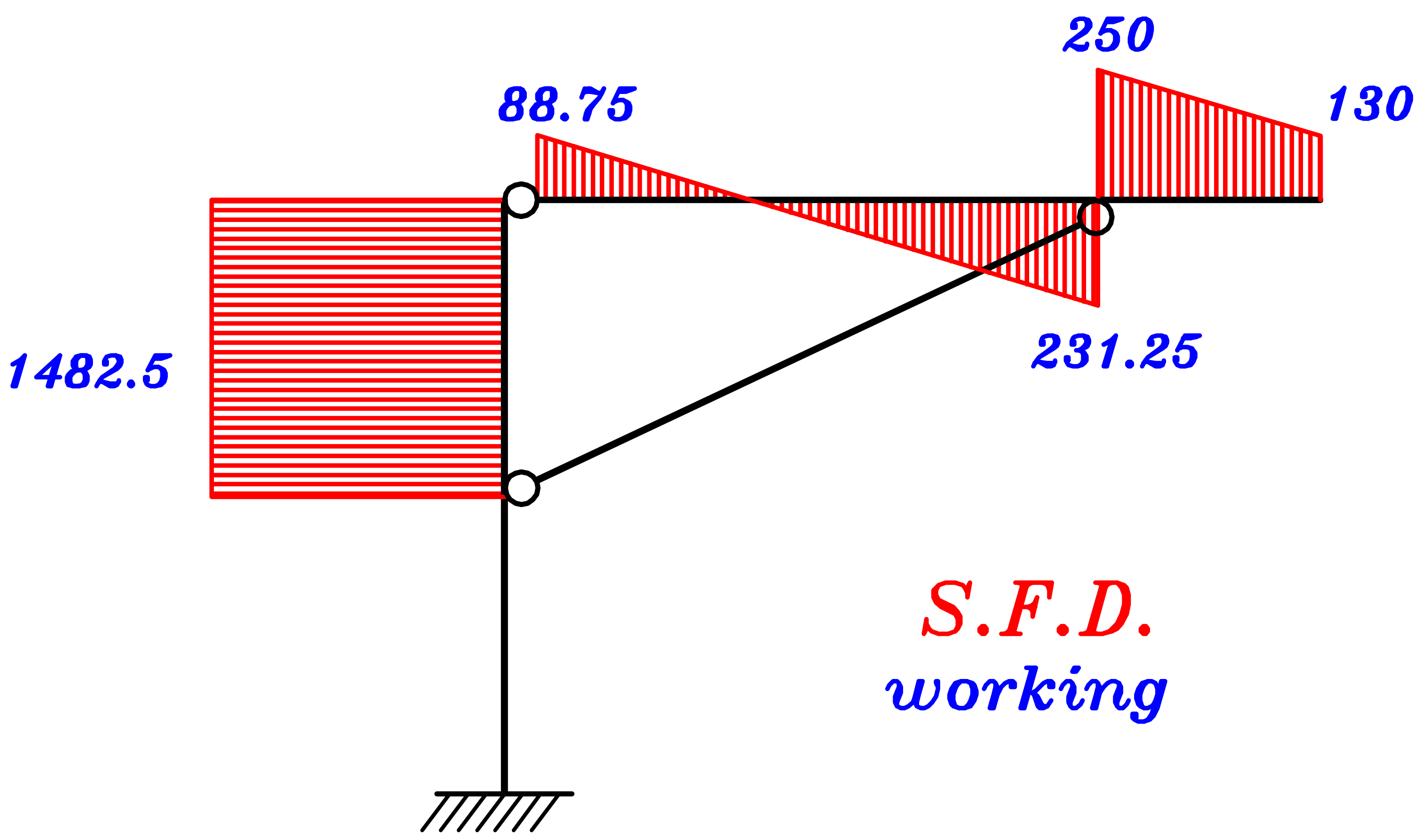
For the Inclined Link

$$\boxed{\frac{Y}{X} = \frac{4}{8} \longrightarrow X = 2Y}$$


195 kN
260 kN
130 kN
40 kN/m
X
Y
4.0
4.0
Y
X
8.0
3.0
1025 kN
5930 kN.m
Y
X
4.0
X
8.0
Y


570
35
5930
B.M.D.
working


1482.5
(+)
283.75
(−)
1657.5
(−)
1025
N.F.D.
working


250
88.75
130
1482.5
231.25
S.F.D.
working

2- *Assuming that the column (a d) is an unbraced column, calculate the bending moments on the column **in & out** of plane due to buckling.*

توجد معلومات ناقصه سنحتاج لفرضها

١- عمق الكمره ال **secondary** $t_{s.b.} = \frac{spacing}{12} = \frac{6.0}{12} = 0.50\ m$

٢- تخانه عمود ال **Frame**

ممكن فرض الابعاد لكن يفضل ان نفرض الابعاد التى نضمن ان تجعل العمود **safe** مع كلا من ال **Bending & Normal**

لذا سنصمم العمود اولا على M_{ext}, P و نحدد الابعاد ثم نعمل **check buckling** على هذه الابعاد .

R-Sec. , $M = 5930 * 1.5 = 8895\ kN.m$, $P = 1025 * 1.5 = 1537.5\ kN$

$$d_o = 3.5\sqrt{\frac{8895 * 10^6}{35 * 450}} = 2630\ mm \qquad (as\ R\text{-}Sec.)$$

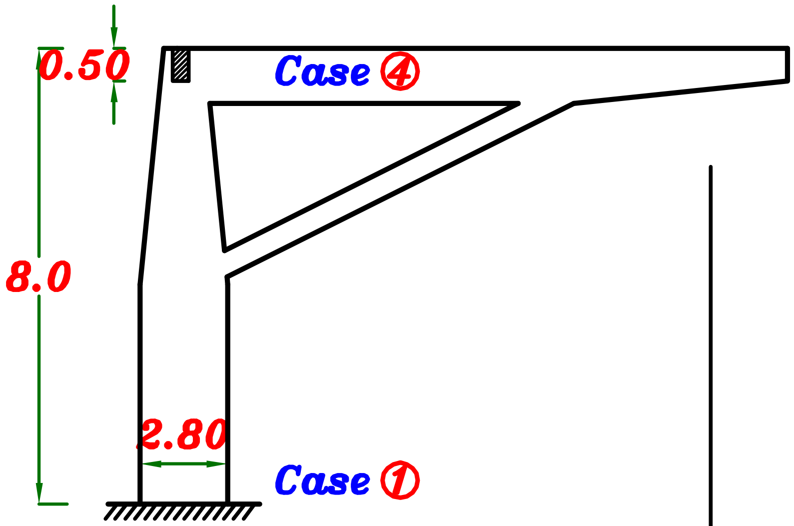
$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(2630) = (2893 \rightarrow 3419)\ mm$$

$\therefore$ **Take** $d = 2900\ mm$, $t = 3000\ mm$

Check $$\frac{P}{F_{cu}\, b\, t} = \frac{1537.5 * 10^3}{35 * 450 * 3000} = 0.032 < 0.04 \quad (Neglect\ P)$$

$\therefore$ **Take** $\boxed{d = 2700\ mm}$, $\boxed{t = 2800\ mm}$

Buckling In plane

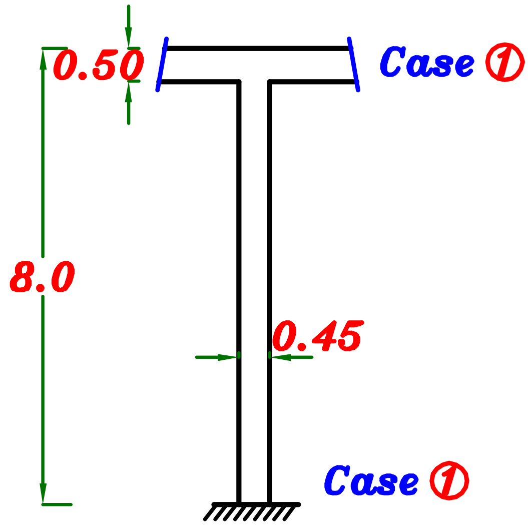


Upper Case ④, Lower Case ① $\Big\}$ $k = 2.2$

$H_o = 8.0\ m$

$$\lambda_{b\,in} = \frac{2.2 * 8.0}{2.8} = 6.28 < 10$$

Buckling Out of plane



Upper Case ①, Lower Case ① $\Big\}$ $k = 1.2$

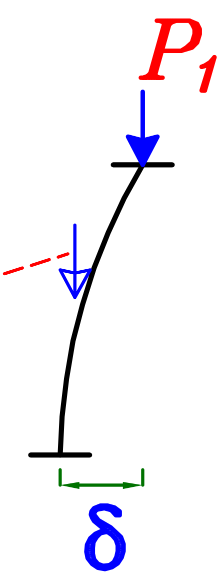
$H_o = 8.0 - 0.5 = 7.50\ m$

$$\lambda_{b\,out} = \frac{1.2 * 7.5}{0.45} = 20 > 10$$

Take the bigger value of $\lambda_b = 20$ (Out of plane)

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{20.0^2 * 0.45}{2000} = 0.09\ m$$

$P_1 = 283.75 * 1.5 = 425.62\ kN$



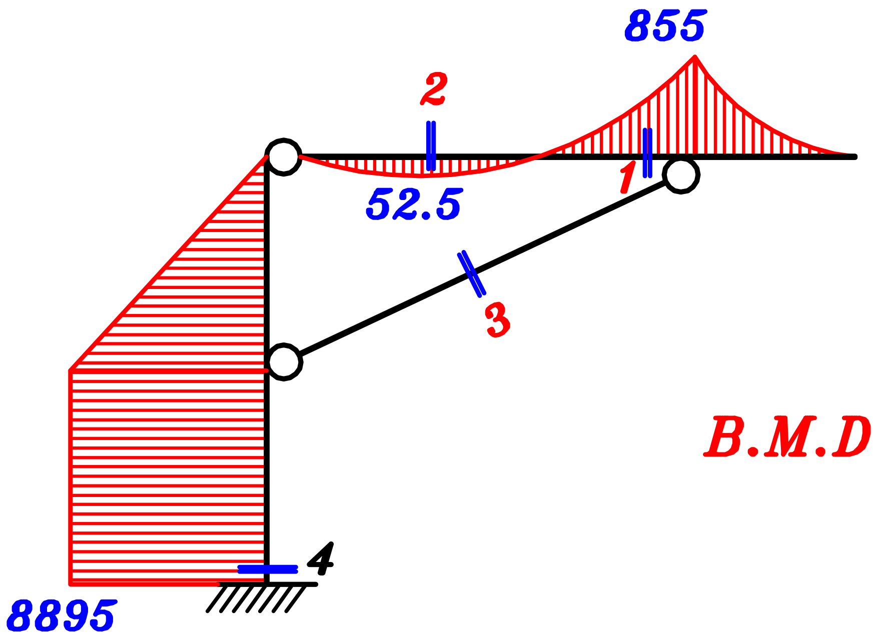
اهمل تأثير ال buckling لهذا الحمل

$$M_{add.} = P_1 * \delta = 425.62 * 0.09 = 38.30\ kN.m$$

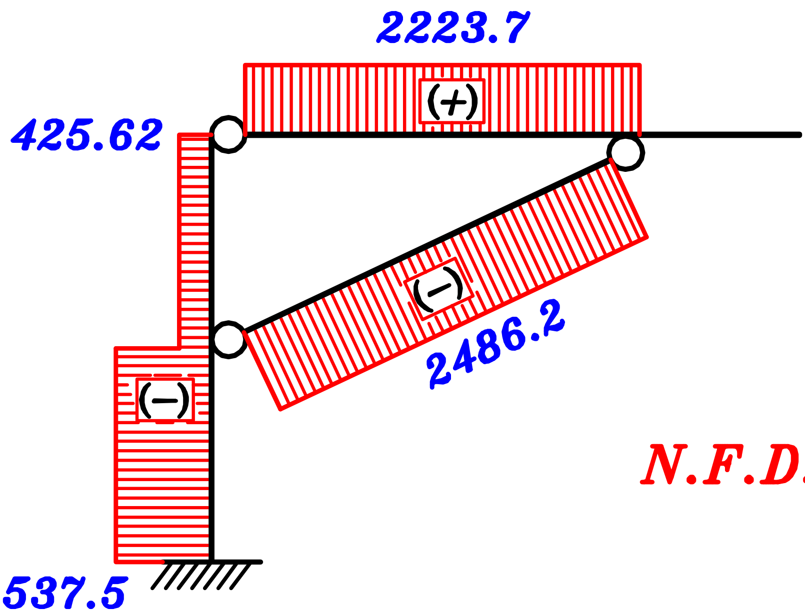
اعلى P و ليس شرط ان تكون اكبر P

3- Design the critical sections (1→4) of the Frame.

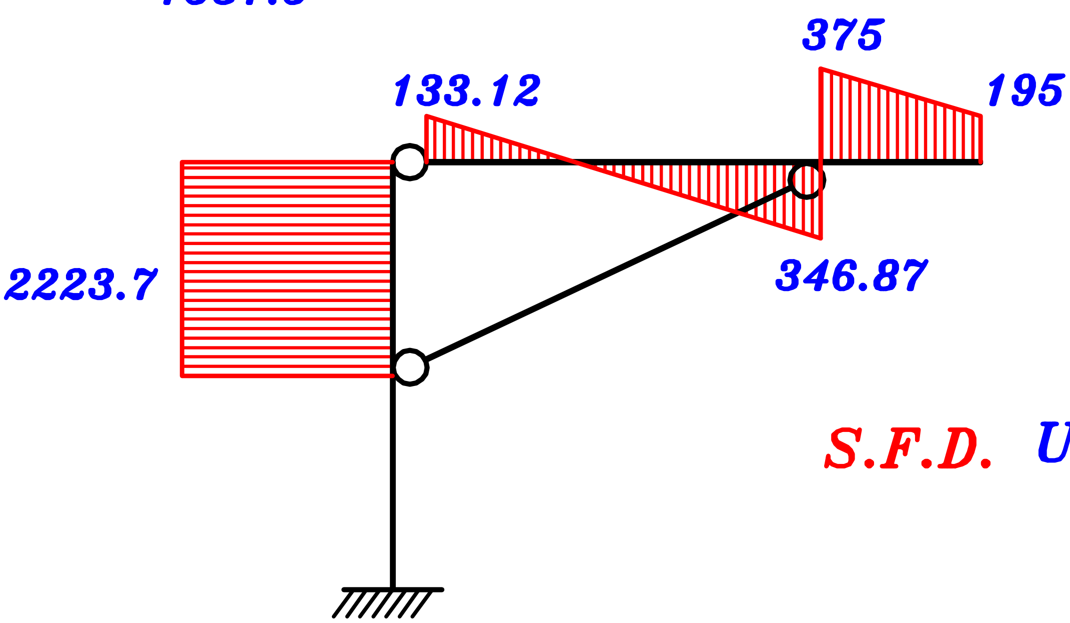
Internal Forces Diagrams



B.M.D. U.L.



N.F.D. U.L.



S.F.D. U.L.

Sec. ① $M = 855\ kN.m$, $T = 2223.7\ kN$, $b = 450\ mm$ **R-Sec.**

$$d_o = 3.5\sqrt{\frac{855 * 10^6}{35 * 450}} = 815.5\ mm$$

$$d = (0.9 \rightarrow 1.0)\ d_o = (0.9 \rightarrow 1.0)\ (815.5) = (733.9 \rightarrow 815.5)\ mm$$

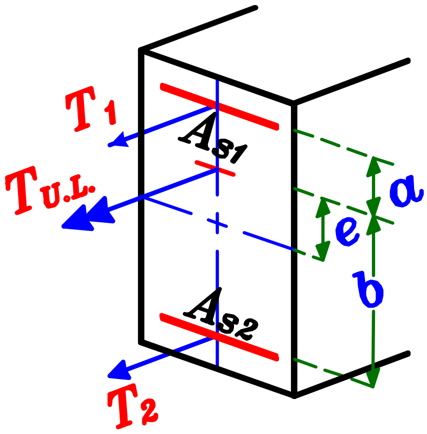
Take $\boxed{d = 850\ mm}$, $\boxed{t = 900\ mm}$

$$e = \frac{M}{T} = \frac{855}{2223.7} = 0.384\ m \quad \therefore \frac{e}{t} = \frac{0.384}{0.90} = 0.42 < 0.5$$

Small Eccentricity.

$$a = \frac{t}{2} - c - e = \frac{0.90}{2} - 0.05 - 0.384 = 0.016\ m$$

$$b = \frac{t}{2} - c + e = \frac{0.90}{2} - 0.05 + 0.384 = 0.784\ m$$



$$\because T_1\ (a+b) = T\ (b)$$

$\boxed{T_2 \text{ بأخذ العزم عند}}$

$$T_1\ (0.80) = 2223.7\ (0.784) \longrightarrow \boxed{T_1 = 2179.2\ kN}$$

$$\because T = T_1 + T_2 \quad \therefore 2223.7 = 2179.2 + T_2 \longrightarrow \boxed{T_2 = 44.5\ kN}$$

$$A_{s1} = \frac{T_1}{(F_y/\gamma_s)} = \frac{2179.2 * 10^3}{(400 \backslash 1.15)} = 6265.2\ mm^2$$

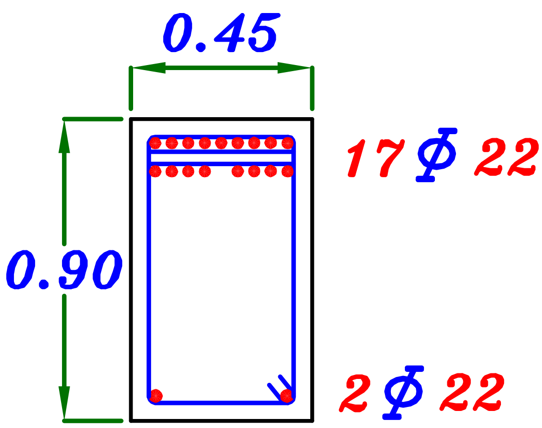
17 Φ 22

$$A_{s2} = \frac{T_2}{(F_y/\gamma_s)} = \frac{44.5 * 10^3}{(400 \backslash 1.15)} = 127.9\ mm^2$$

2 Φ 22

$$\therefore n = \frac{b - 25}{\phi + 25}$$

$$= \frac{450 - 25}{22 + 25} = 9.04 = 9.0$$



Sec. ②

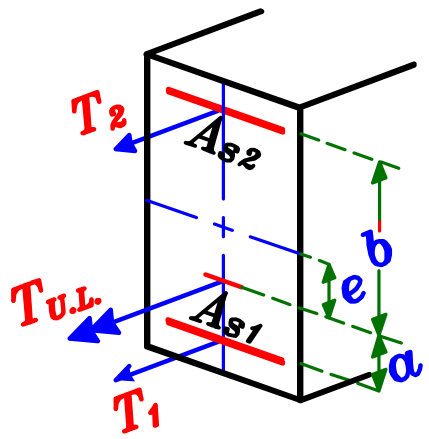
$M = 52.5\ kN.m$, $T = 2223.7\ kN$, $b = 450\ mm$ R-Sec.

$$e = \frac{M}{T} = \frac{52.5}{2223.7} = 0.023\ m \quad \therefore \frac{e}{t} = \frac{0.023}{0.90} = 0.026 < 0.5$$

Small Eccentricity.

$$a = \frac{t}{2} - c - e = \frac{0.90}{2} - 0.05 - 0.023 = 0.377\ m$$

$$b = \frac{t}{2} - c + e = \frac{0.90}{2} - 0.05 + 0.023 = 0.423\ m$$



$$\because\ T_1\,(a+b) = T\,(b)$$

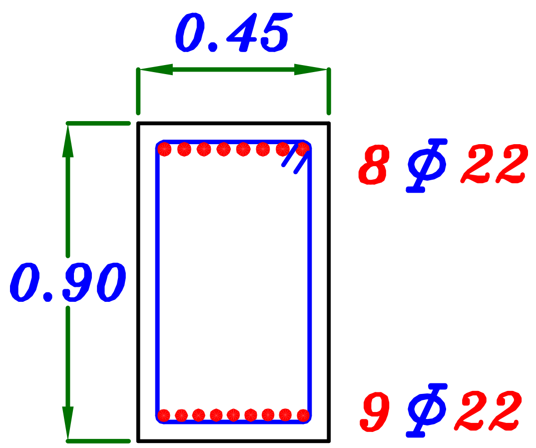
$\boxed{T_2 \text{ بأخذ العزم عند}}$

$$T_1\,(0.80) = 2223.7\,(0.423) \longrightarrow \boxed{T_1 = 1175.8\ kN}$$

$$\because\ T = T_1 + T_2 \quad \therefore 2223.7 = 1175.8 + T_2 \longrightarrow \boxed{T_2 = 1047.9\ kN}$$

$$A_{s1} = \frac{T_1}{(F_y/\gamma_s)} = \frac{1175.8 * 10^3}{(400\backslash 1.15)} = 3380.4\ mm^2 \qquad 9\ \phi\ 22$$

$$A_{s2} = \frac{T_2}{(F_y/\gamma_s)} = \frac{1047.9 * 10^3}{(400\backslash 1.15)} = 3012.7\ mm^2 \qquad 8\ \phi\ 22$$



Sec. ③ $P = 2486.2\ kN$, $(b * \frac{t}{2}) = (450 * 450)$

Neglect effect of buckling

$A_c = 450 * 450 = 202500\ mm^2$

$\because P_{U.L.} = 0.35\ A_c\ F_{cu} + 0.67\ A_s\ F_y$

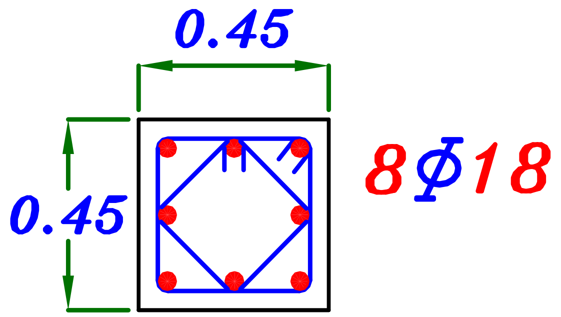
$\therefore 2486.2 * 10^3 = 0.35\ (202500)\ (35) + 0.67\ A_s\ (400)$

$\therefore A_s = 20.80\ mm^2 < A_{s\,min}$

$A_{s\,min} = \frac{0.8}{100} * A_c = \frac{0.8}{100} * 202500 = 1620\ mm^2$

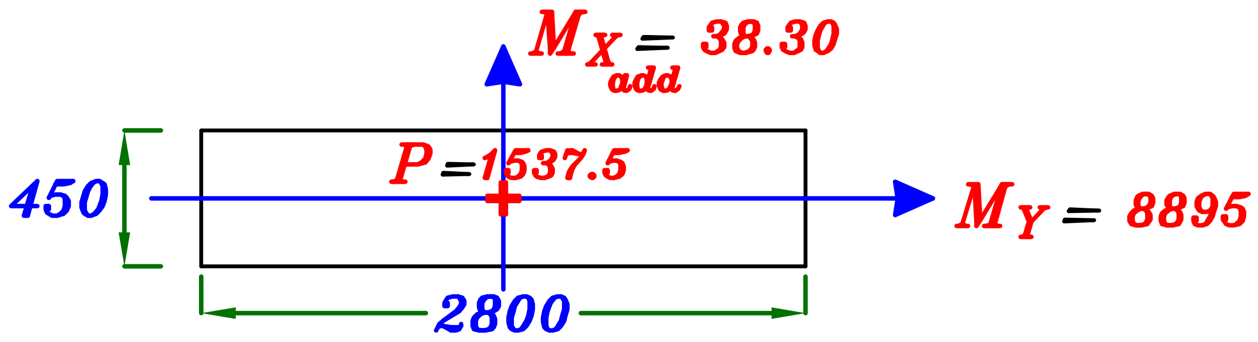
$\therefore A_s < A_{s\,min} \longrightarrow A_s = A_{s\,min} = 1620\ mm^2$

$\boxed{8\phi18}$



Sec. ④ $b = 450\ mm$, $t = 2800\ mm$ R-Sec.

$P = 1537.5\ kN.m$ $M_Y = 8895\ kN.m$ $M_{X_{add}} = 38.30\ kN.m$



لان قيمة M_X اقل كثيرا من M_Y فمن الممكن فى الامتحان اهمال قيمة M_X

Neglect M_X because it is too small.

Design the section on :

$P = 1537.5$ kN.m , $M_Y = 8895$ kN.m

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{1537.5 * 10^3}{35 * 450 * 2800} = 0.035 < 0.04$ (Neglect P)

$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu} B}} \quad \therefore 2700 = C_1 \sqrt{\frac{8895 * 10^6}{35 * 450}} \longrightarrow C_1 = 3.60 \longrightarrow J = 0.786$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{8895 * 10^6}{0.786 * 400 * 2700} = 10478 \text{ mm}^2$$

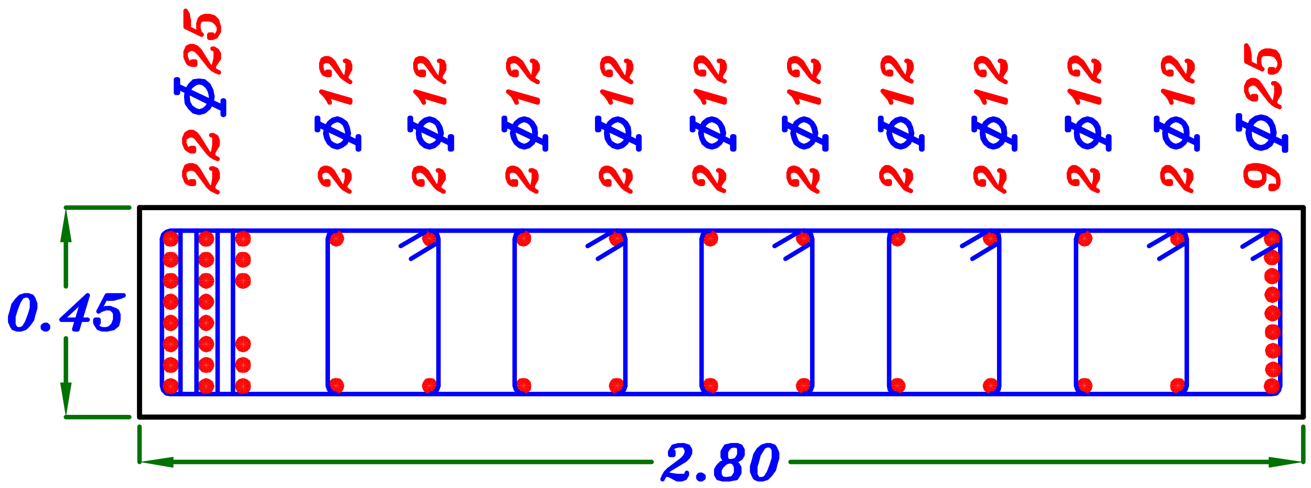
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 10478 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{400}\right) 450 * 2800 = 4193 \text{ mm}^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 10478 \text{ mm}^2 \qquad 22\Phi25$$

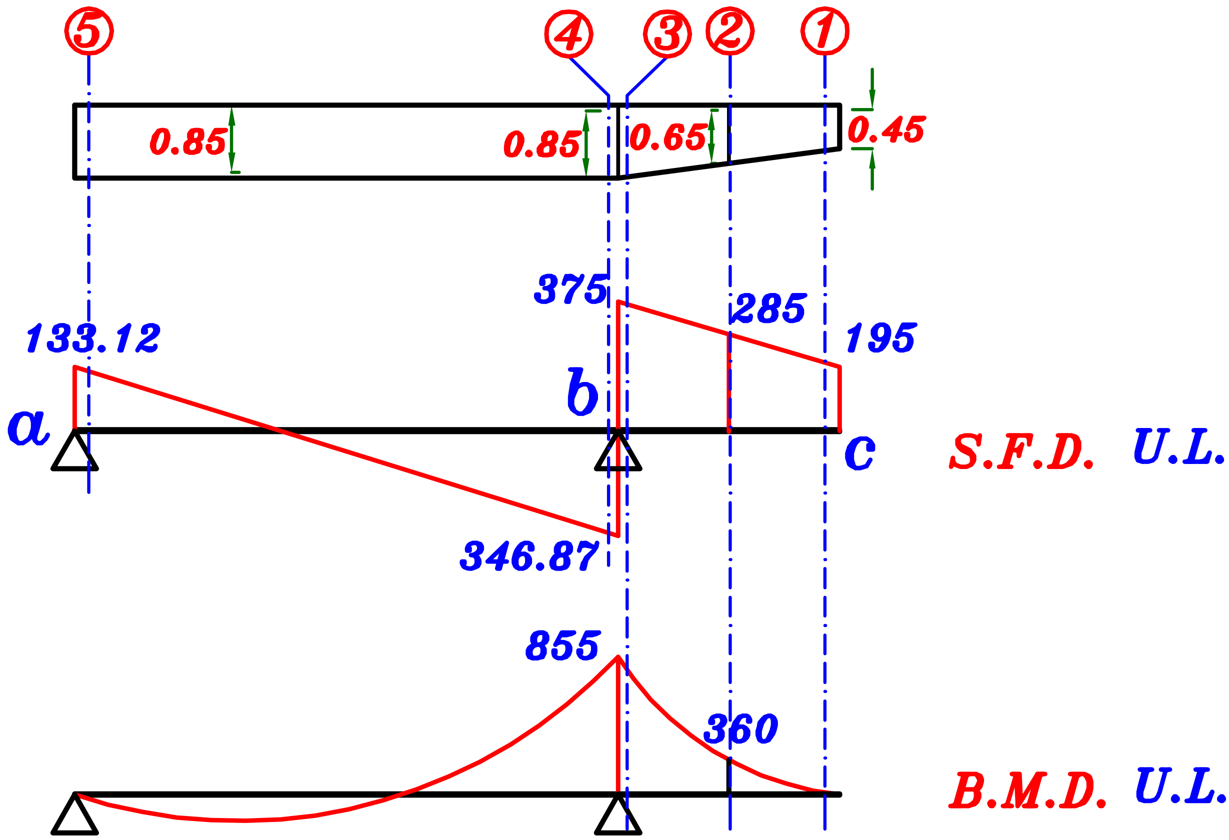
$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{450 - 25}{25 + 25} = 8.50 = 8.0 \text{ bars}$$

Stirrup Hangers $= 0.4\, A_s = 0.4\,(10478) = 4191.2 \text{ mm}^2 \qquad 9\Phi25$



4- Draw the shear stress diagram of members a-b-c and design its critical sections For shear.

لرسم ال Shear Stress Diagram سنحسب ال stress عند ثلاث نقط للجزء المائل
و نوصل بينهم بمنحنى و نقطتين عند الجزء الافقى و نوصل بينهم بخط.



For inclined part.

$$\tan\beta = \frac{0.9-0.5}{3.0} = 0.133$$

$$q_U = \frac{Q}{b\,d} - \frac{M\tan\beta}{b\,d^2}$$

Point 1

$Q = 195$ kN. $M = \text{Zero}$ kN.m $d = 450$ mm

$$q = \frac{195 * 10^3}{450 * 450} - \text{Zero} = 0.963 \ N\backslash mm^2$$

Point 2

$Q = 285$ kN. $M = 360$ kN.m $d = 650$ mm

$$q = \frac{285 * 10^3}{450 * 650} - \frac{360 * 10^6 * (0.133)}{450 * 650^2} = 0.722 \ N\backslash mm^2$$

Point ③

$Q = 375$ kN. $M = 855$ kN.m $d = 850$ mm

$$q = \frac{375 * 10^3}{450 * 850} - \frac{855 * 10^6 * (0.133)}{450 * 850^2} = 0.630 \ N\backslash mm^2$$

For straight part.

$$\boxed{q_U = \frac{Q}{b\,d}}$$

Point ④

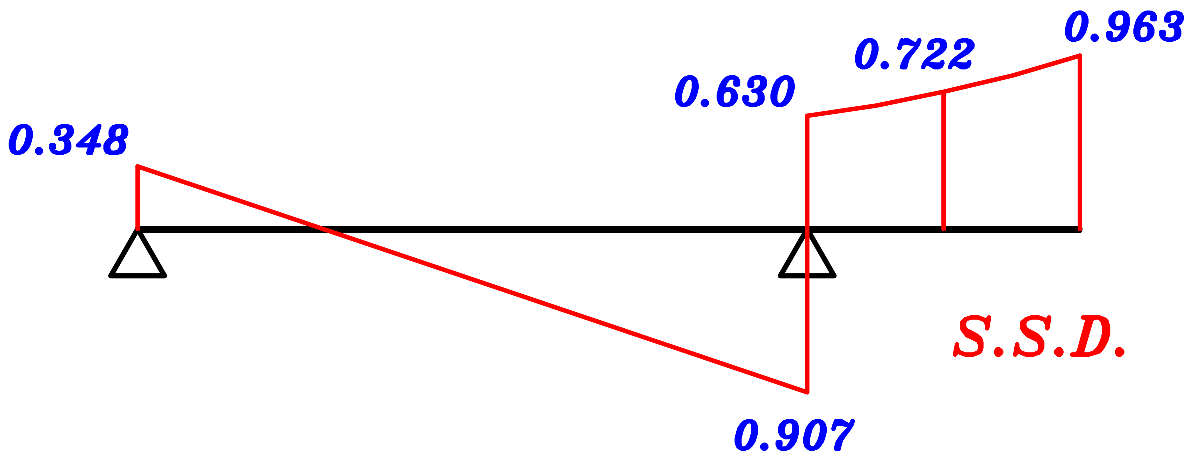
$Q = 346.87$ kN. $d = 850$ mm

$$q = \frac{346.87 * 10^3}{450 * 850} = 0.907 \ N\backslash mm^2$$

Point ⑤

$Q = 133.12$ kN. $d = 850$ mm

$$q = \frac{133.12 * 10^3}{450 * 850} = 0.348 \ N\backslash mm^2$$



S.S.D.

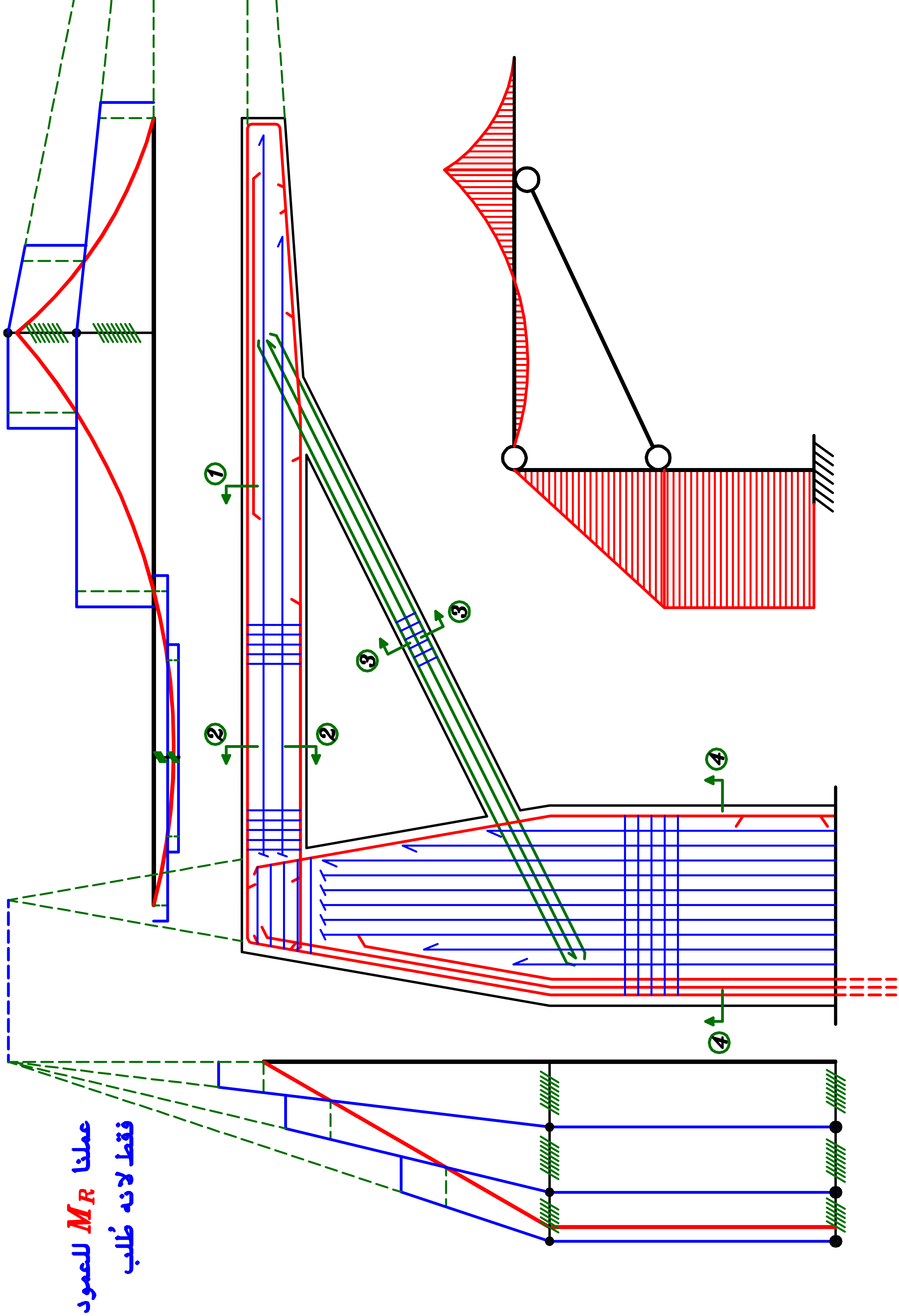
Check Shear.

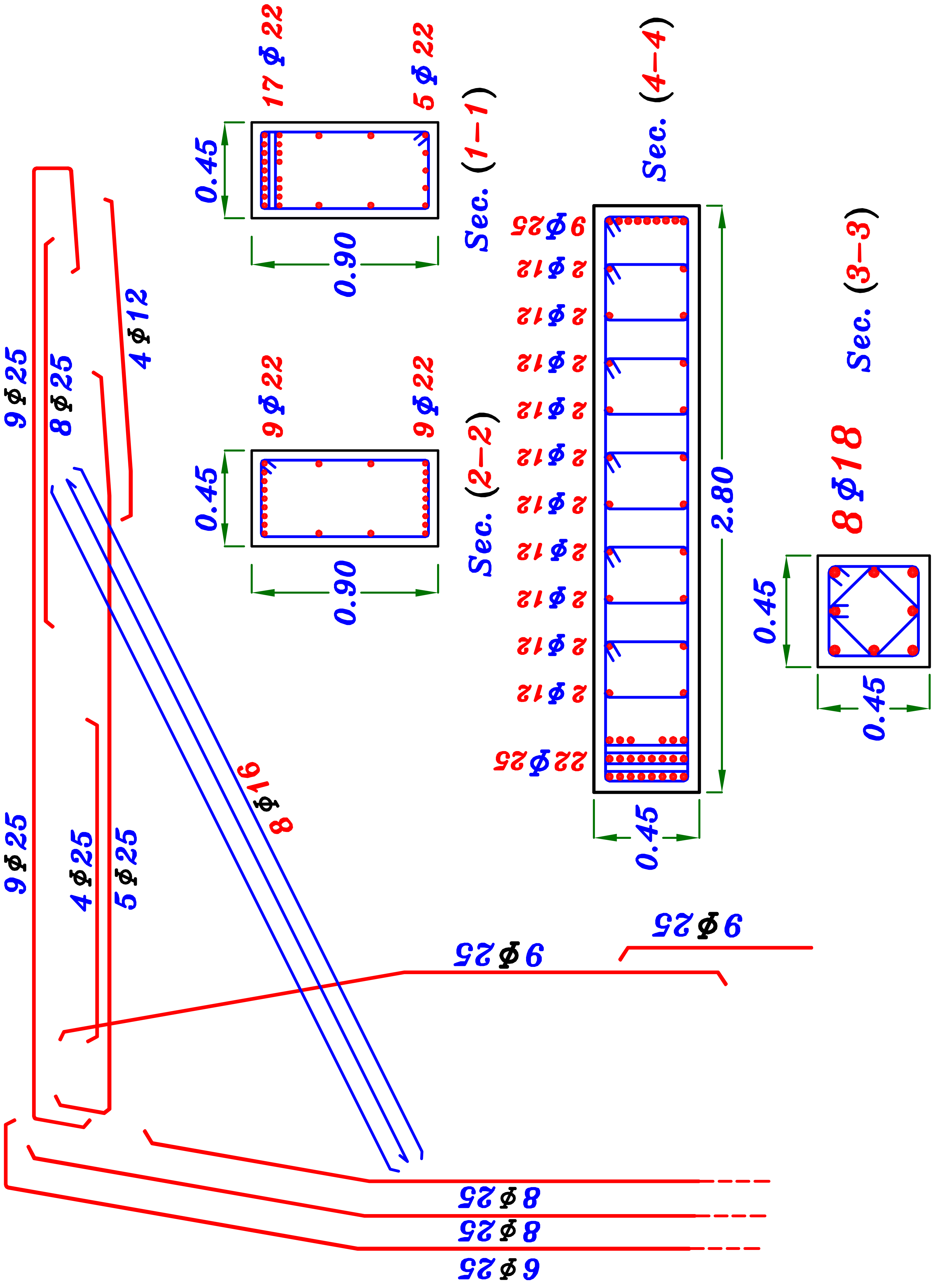
– Allowable shear stress.

$$q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{35}{1.5}} = 1.16 \ N\backslash mm^2$$

∵ All sections have stresses less than q_{cu}

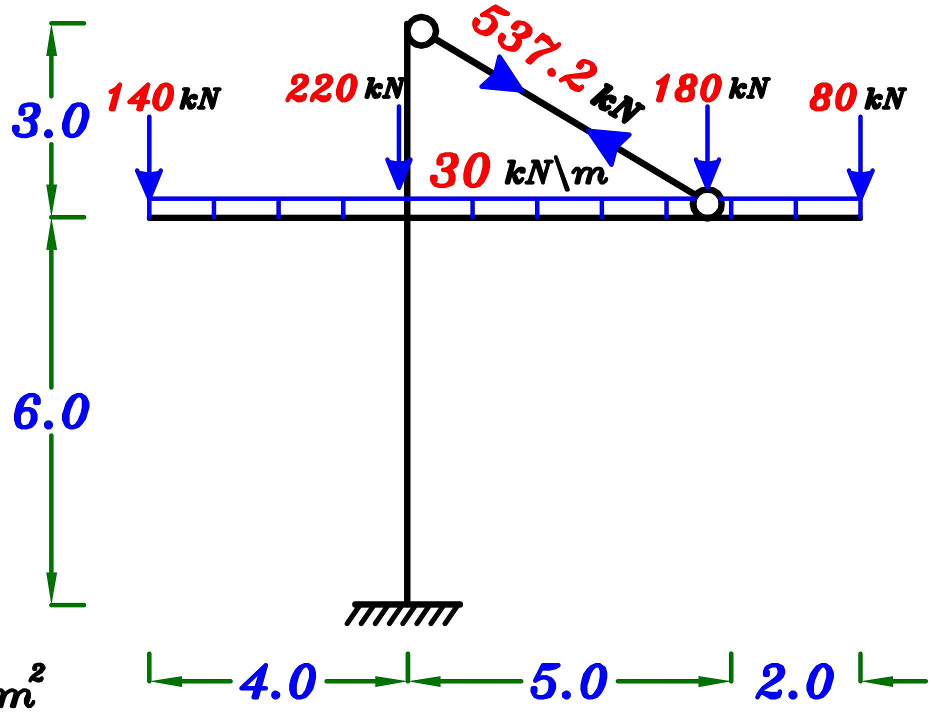
∴ Stirrups For all sections are $\boxed{5\phi 8\backslash m'}$ 2 branches


1
2 2
3 3
4 4
عملنا MR للعمود
فقط لانه طُلب



Sec. (1-1)
17Φ22
5Φ22
0.45
0.90
Sec. (2-2)
9Φ22
9Φ22
0.45
0.90
Sec. (4-4)
9Φ25
2Φ12
22Φ25
2.80
0.45
Sec. (3-3)
8Φ18
0.45
0.45
9Φ25
8Φ25
4Φ12
9Φ25
4Φ25
5Φ25
8Φ16
9Φ25
9Φ25
8Φ25
8Φ25
6Φ25

Example.



Data.

$b_{(Frame)} = 350$ mm

st. 360/520

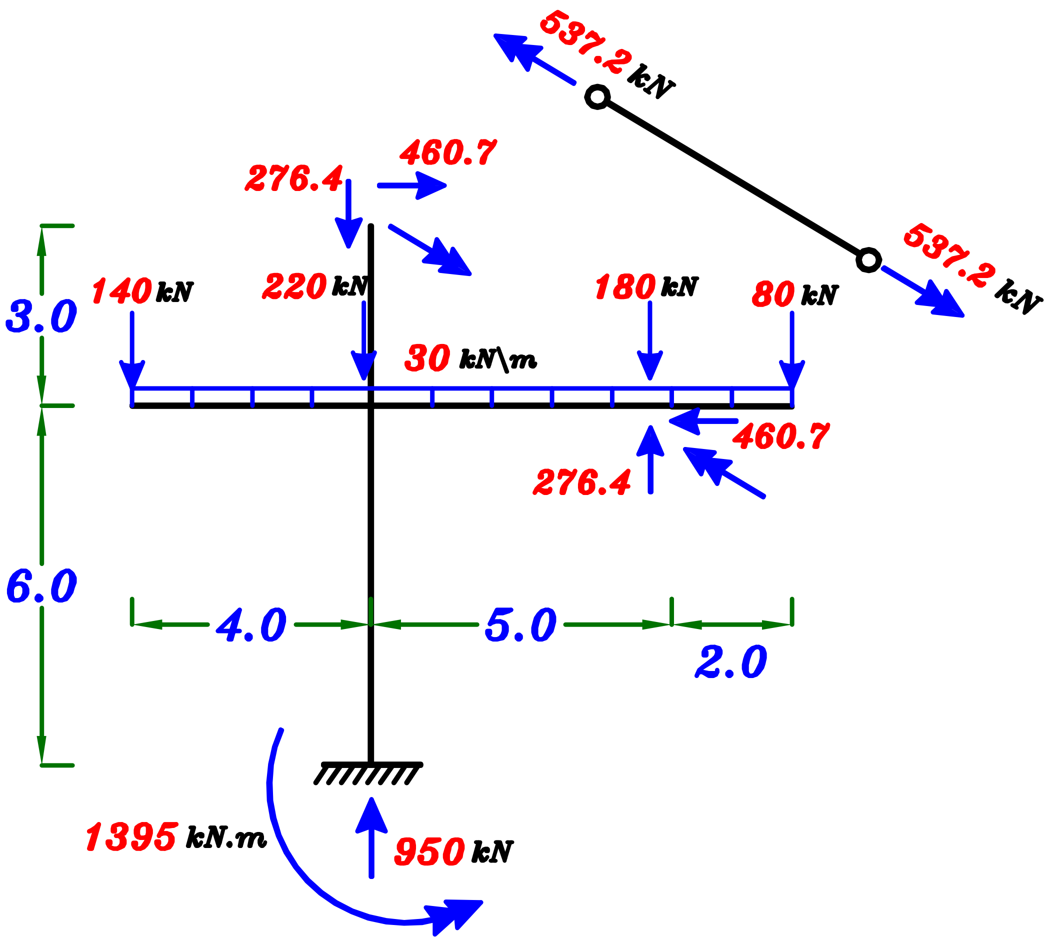
$C = 30 = F_{cu} = 30$ N\mm^2

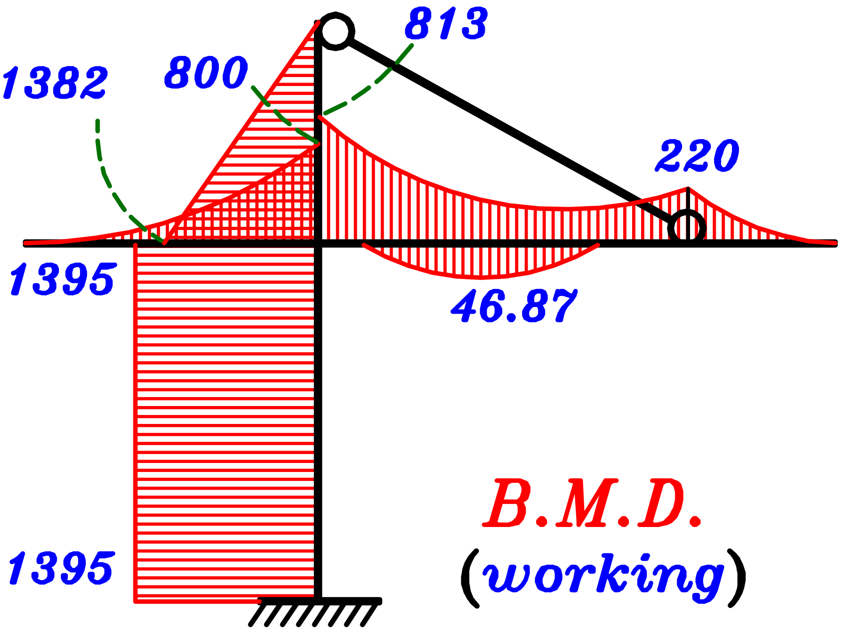
Req.

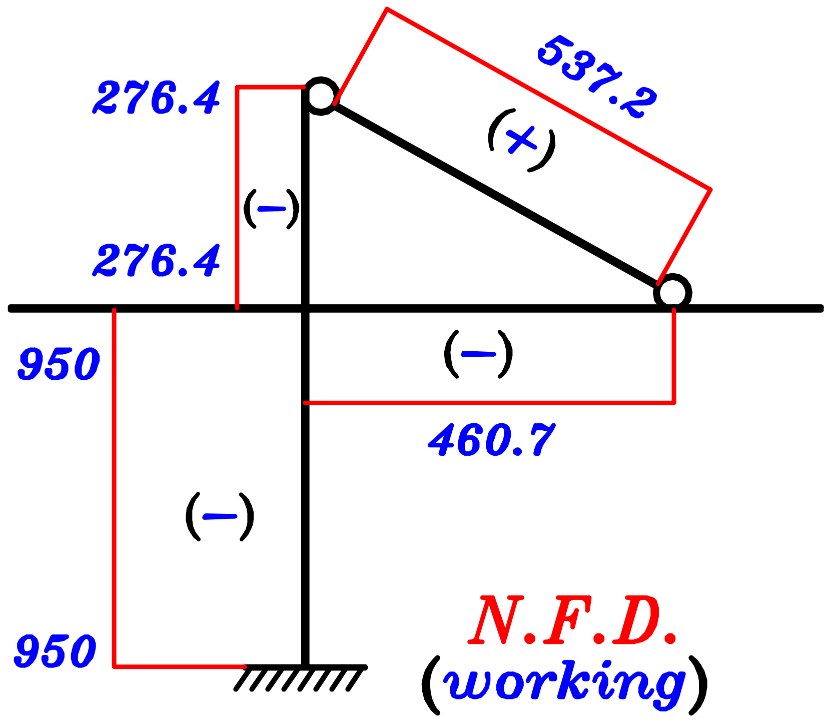
1 - Draw B.M.D. , N.F.D. & S.F.D.

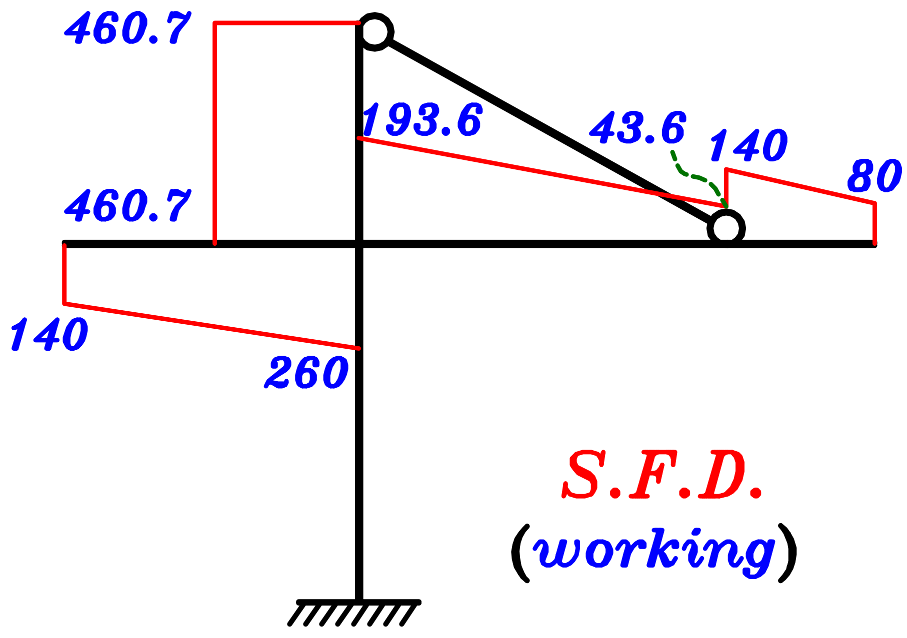
2 - Design the critical sections of the Frame For Bending and shear.

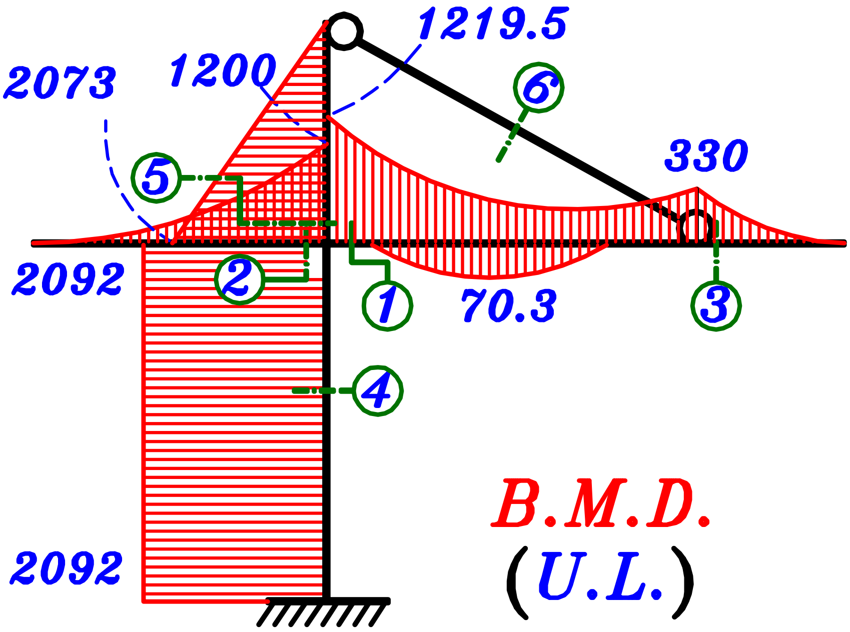
3 - Draw the details of reinforcement For intermediate Frame (F) in elevation to scale 1:50 and cross sections to scale 1:25

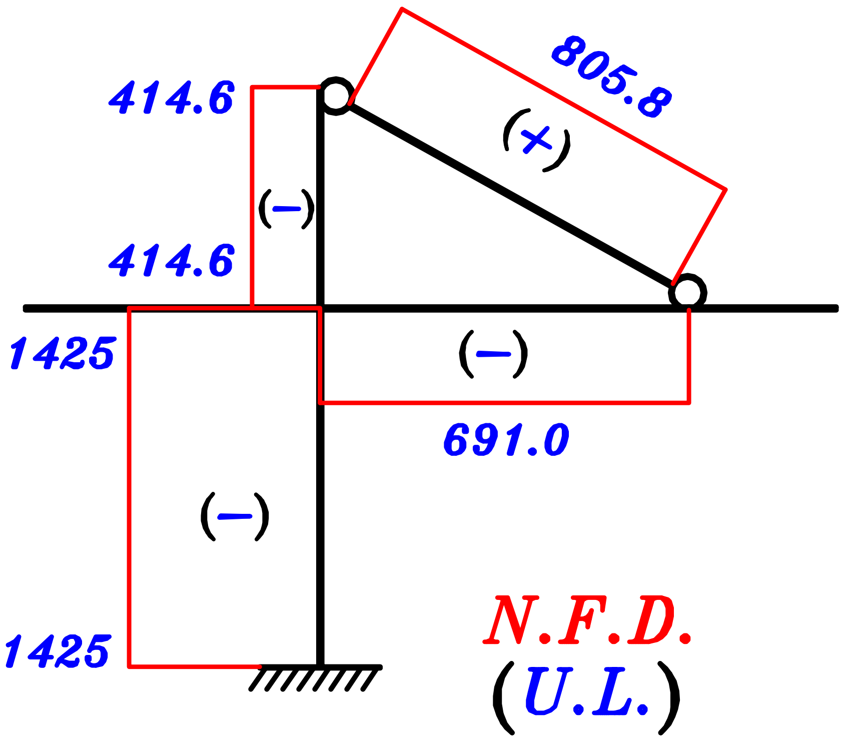


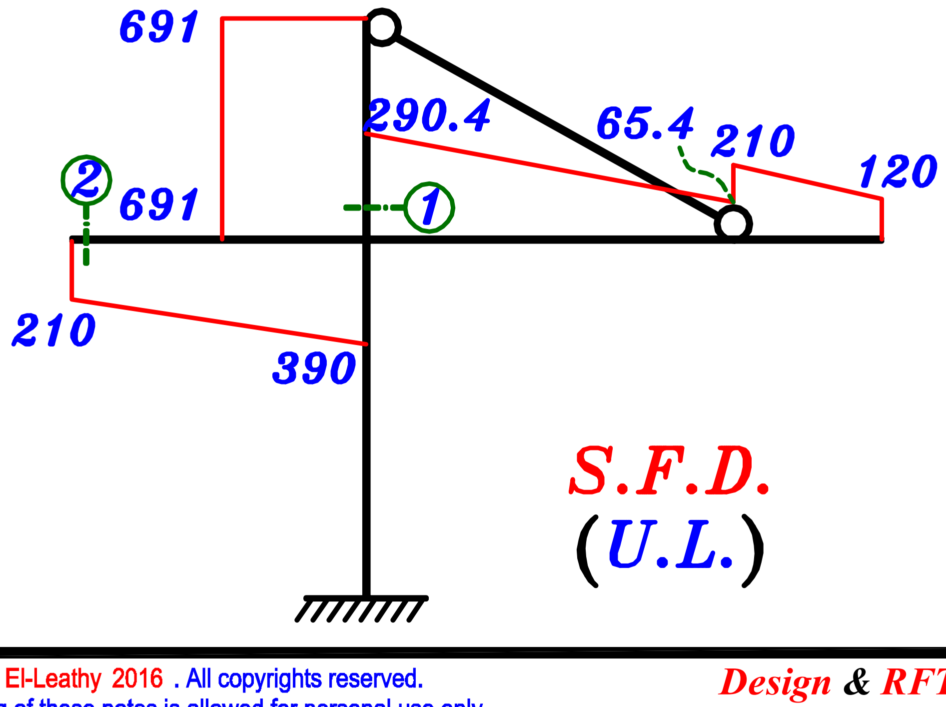

1382
800
813
220
1395
46.87
1395
B.M.D.
(working)


537.2
276.4
(+)
(–)
276.4
950
(–)
460.7
(–)
950
N.F.D.
(working)


460.7
193.6
43.6
140
80
460.7
140
260
S.F.D.
(working)


2073
1200
1219.5
6
5
330
2092
2
1
70.3
3
4
2092
B.M.D.
(U.L.)


414.6
805.8
(+)
(−)
414.6
1425
(−)
691.0
(−)
1425
N.F.D.
(U.L.)


691
290.4
65.4
210
120
2
691
1
210
390
S.F.D.
(U.L.)

2 - Design the critical sections of the Frame For Bending and shear.

Sec. ① $M = 1219.5$ kN.m , $P = 691.0$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{1219.5 * 10^6}{30 * 350}} = 1192.8 \text{ mm} \quad \text{(as R-Sec.)}$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1192.8) = (1312.1 \rightarrow 1550.6) \text{ mm}$$

$\therefore$ Take $\boxed{d = 1400 \text{ mm}}$, $\boxed{t = 1500 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{691.0 * 10^3}{30 * 350 * 1500} = 0.0438 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{1219.5}{691.0} = 1.76 \text{ m} \quad \therefore \frac{e}{t} = \frac{1.76}{1.5} = 1.17 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.76 + \frac{1.50}{2} - 0.10 = 2.41 \text{ m}$$

$$M_s = P * e_s = 691.0 * 2.41 = 1665.3 \text{ kN.m}$$

$$\therefore 1400 = C_1\sqrt{\frac{1665.3 * 10^6}{30 * 350}} \longrightarrow C_1 = 3.51 \longrightarrow J = 0.78$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{N_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{1665.3 * 10^6}{0.780 * 350 * 1400} - \frac{691.0 * 10^3}{(360 \backslash 1.15)} = 2149.8 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2149.8 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1400 = 1677.4 \text{ mm}^2$$

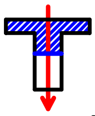
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2149.8 \text{ mm}^2$ $\quad$ 6 ϕ22

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{22 + 25} = 6.91 = 6.0 \text{ bars}$$

Sec. ② $M = 1200$ kN.m , $P = Zero$ kN , $b = 350$ mm

$d = 1400$ mm (the same depth of sec. ①)

The sec. is R-sec.



$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 1400 = C_1 \sqrt{\frac{1200 * 10^6}{30 * 350}} \longrightarrow C_1 = 4.14 \longrightarrow J = 0.808$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1200 * 10^6}{0.808 * 360 * 1400} = 2946.7 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2946.7 \text{ mm}^2$

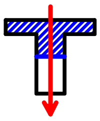
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1400 = 1677.4 \text{ mm}^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 2946.7 \text{ mm}^2 \quad 8 \phi 22$$

Sec. ③ $M = 330$ kN.m , $P = Zero$ kN , $b = 350$ mm

$d = 1400$ mm (the same depth of sec. ①)

The sec. is R-sec.



$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 1400 = C_1 \sqrt{\frac{330 * 10^6}{30 * 350}} \longrightarrow C_1 = 7.89 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{330 * 10^6}{0.826 * 360 * 1400} = 792.7 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 792.7 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1400 = 1677.4 \text{ mm}^2$$

$$\mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \dfrac{\sqrt{30}}{360}\right) 350 * 1400 = 1677.4 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 792.7 = 1030.5 \text{ mm}^2 \end{array}\right\} \text{الأقل} = 1030.5 \text{ mm}^2 \quad 3 \phi 22$$

Sec. ④ $M = 2092$ kN.m , $P = 1425$ kN , $b = 350$ mm

$$d_{\circ} = 3.5\sqrt{\frac{2092 * 10^{6}}{30 * 350}} = 1562.2 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_{\circ} = (1.1 \rightarrow 1.3)\,(1562.2) = (1718.5 \rightarrow 2030) \text{ mm}$$

$\therefore$ Take $\boxed{d = 1800 \text{ mm}}$, $\boxed{t = 1900 \text{ mm}}$

Check $$\frac{P}{F_{cu}\, b\, t} = \frac{1425 * 10^{3}}{30 * 350 * 1900} = 0.071 > 0.04 \quad (\text{Don't neglect } P)$$

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{2092}{1425} = 1.47 \text{ m} \quad \therefore \frac{e}{t} = \frac{1.47}{1.90} = 0.77 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.47 + \frac{1.90}{2} - 0.10 = 2.32 \text{ m}$$

$$M_s = P * e_s = 1425 * 2.32 = 3306 \text{ kN.m}$$

$$\therefore 1800 = C_1\sqrt{\frac{3306 * 10^{6}}{30 * 350}} \longrightarrow C_1 = 3.20 \longrightarrow J = 0.76$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{3306 * 10^{6}}{0.780 * 360 * 1800} - \frac{1425 * 10^{3}}{(360 \backslash 1.15)} = 1988.7 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1988.7 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1800 = 2156.6 \text{ mm}^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$\left.\begin{aligned} A_{s_{min.}} &= \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1800 = 2156.6 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} &= 1.3 * 1988.7 = 2585.3 \text{ mm}^2 \end{aligned}\right\} \text{الأقل} = 2156.6 \text{ mm}^2 \quad 6\,\phi\,22$$

Sec. ⑤ $M = 2073$ kN.m , $P = 414.6$ kN , $b = 350$ mm

(Take the same depth of Sec. ④)

$\therefore$ Take $\boxed{d = 1800 \text{ mm}}$, $\boxed{t = 1900 \text{ mm}}$

Check $\frac{P}{F_{cu}\, b\, t} = \frac{414.6 * 10^3}{30 * 350 * 1900} = 0.020 < 0.04$ (Neglect P)

$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 1800 = C_1 \sqrt{\frac{2073 * 10^6}{30 * 350}} \longrightarrow C_1 = 4.05 \longrightarrow J = 0.805$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{2073 * 10^6}{0.805 * 360 * 1800} = 3974 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 3974 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1800 = 2156.6 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3974 \text{ mm}^2$ $\quad$ 11 ϕ 22

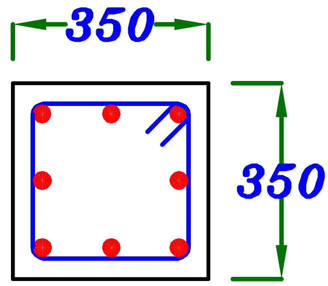
Sec. ⑥ $T = 805.8$ kN

لانه Link member و عليه قوى Tension فقط
فلن يحتاج الى ابعاد كبيره فمن الممكن اخذ الابعاد $(b * b)$

$$A_c = (b * b) \quad (350 * 350)$$

$$A_s = \frac{T}{F_y \backslash \gamma_s} = \frac{805.8 * 10^3}{360 \backslash 1.15} = 2574.7 \text{ mm}^2$$

8 ϕ 22



8 ϕ 22

Check Shear.

- Allowable shear stress.

- $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{30}{1.5}} = 1.07 \ N\backslash mm^2$

- $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{30}{1.5}} = 3.0 \ N\backslash mm^2$

Sec. ① $Q = 691$ kN

$\therefore$ Actual shear stress. $= q_U = \frac{Q}{b\,d} = \frac{691 * 10^3}{350 * 1800} = 1.09 \ N\backslash mm^2$

$\therefore q_{cu} < q_U < q_{max.} \therefore$ We need Stirrups more Than $5\phi 8\backslash m`$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n\,A_s (F_y\backslash \gamma_s)}{b\;S}$

* Take $n = 2$, $\phi 8 \longrightarrow A_s = 50.3 \ mm^2$

$1.09 - \frac{1.07}{2} = \frac{2 * 50.3\,(240\backslash 1.15)}{350 * S} \longrightarrow S = 108.1 \ mm > 100 \ mm \ \therefore$ o.k.

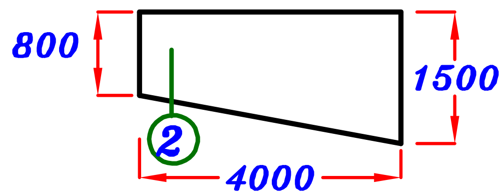
$\therefore$ No. of stirrups$\backslash m`$ $= \frac{1000}{S} = \frac{1000}{108.1} = 9.25 = 10$

$\therefore$ Use Stirrups $\boxed{10\phi 8\backslash m`}$ 2 branches

Sec. ② $Q = 210$ kN

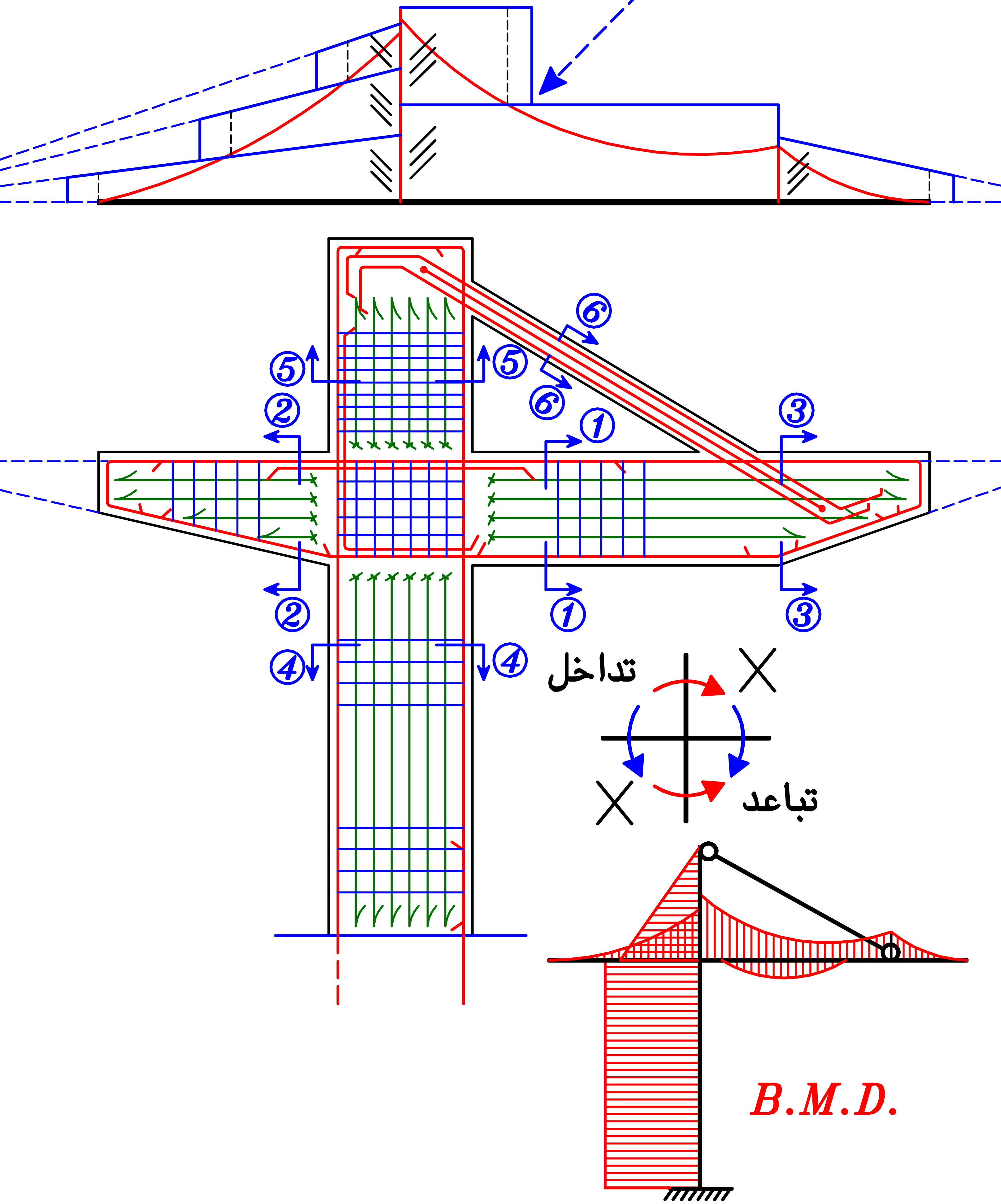
Actual shear stress. $= q_U = \frac{Q}{b\,d} - \frac{M\,\tan\beta}{b\;d^2}$

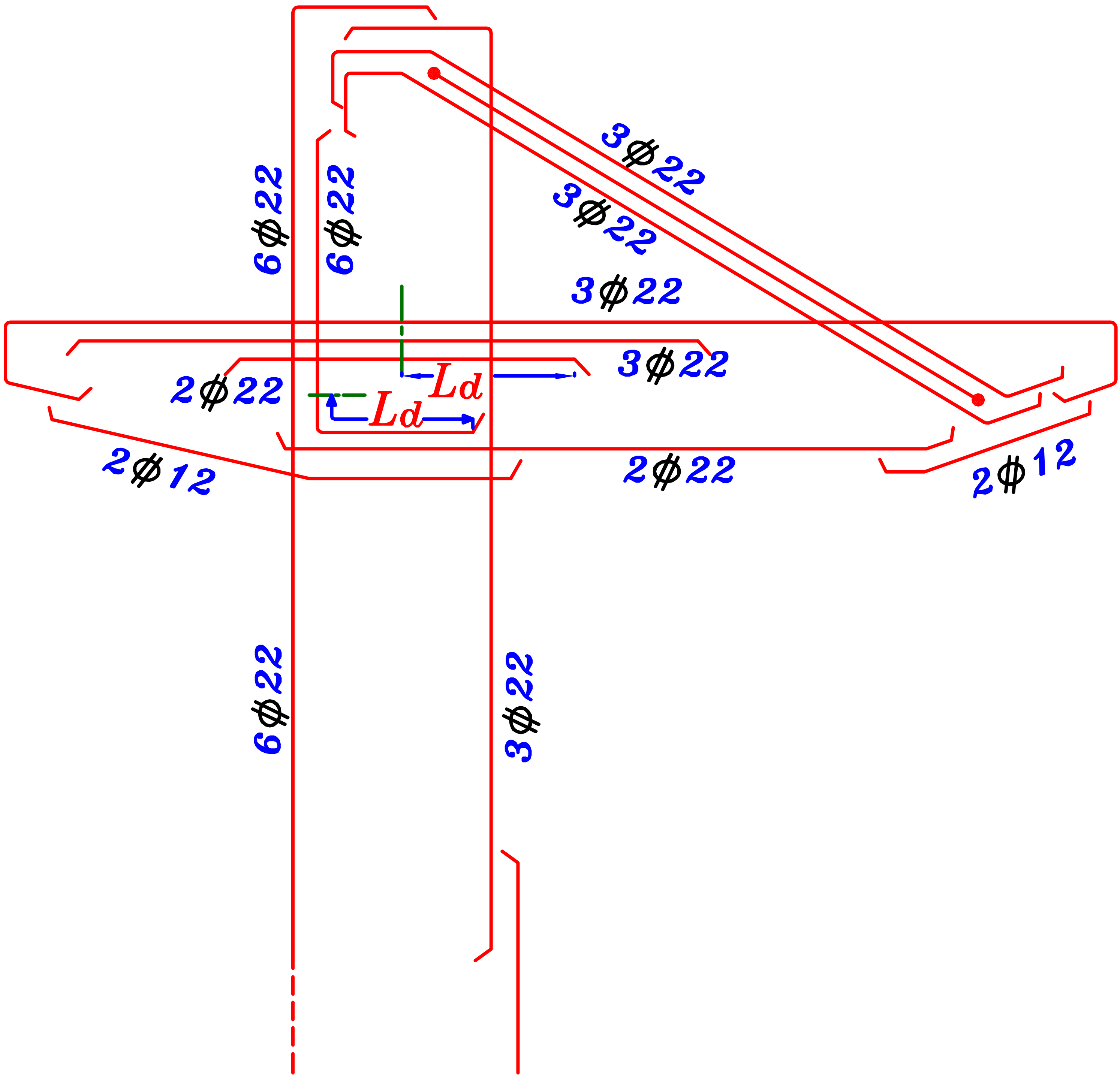
$q_U = \frac{210 * 10^3}{350 * 750} - ZERO = 0.80 \ N\backslash mm^2$

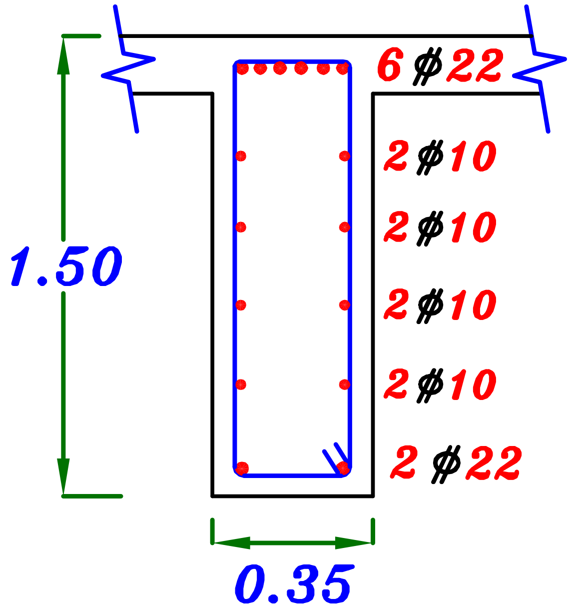


$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8\backslash m`}$

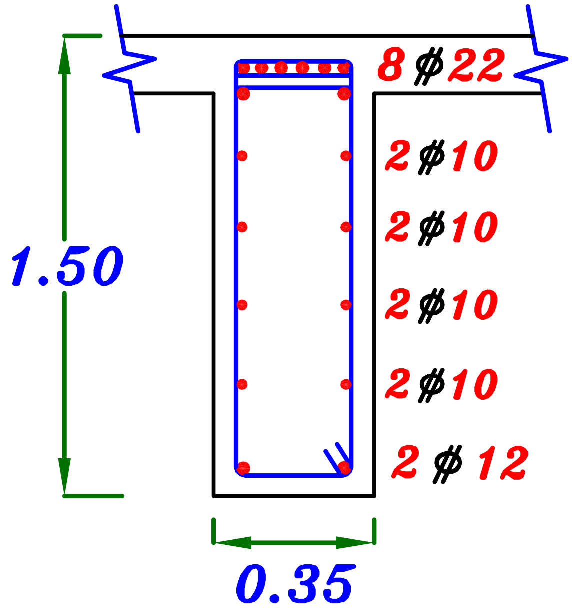
ملحوظه scale البلوكات مختلف فى هذه المنطقه
لوجود Normal



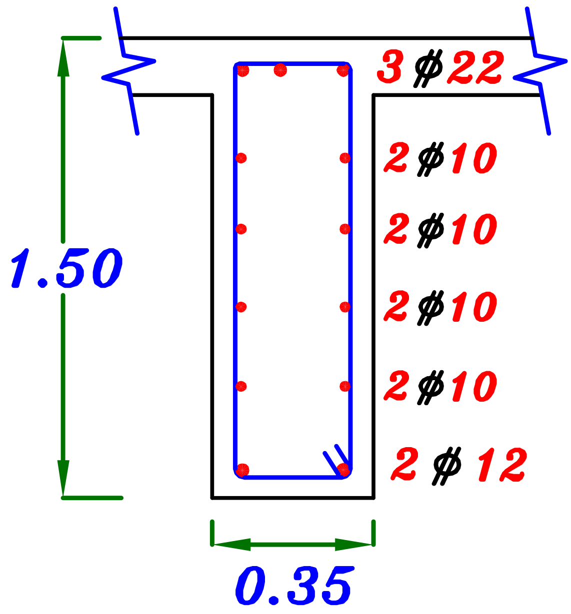

3Ø22
3Ø22
6Ø22
6Ø22
3Ø22
3Ø22
2Ø22
Ld
Ld
2Ø12
2Ø22
2Ø12
6Ø22
3Ø22



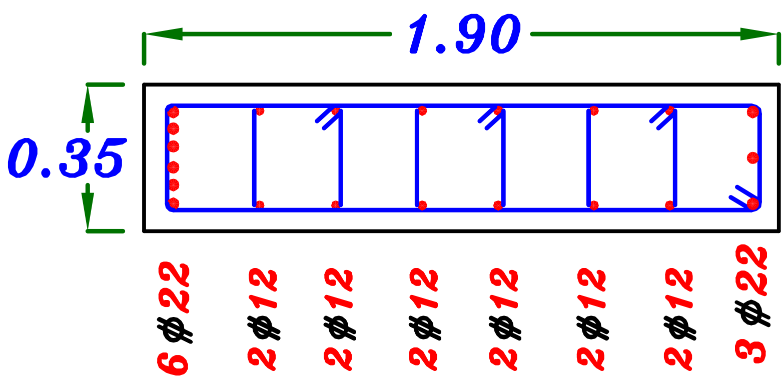
Sec. (1–1)



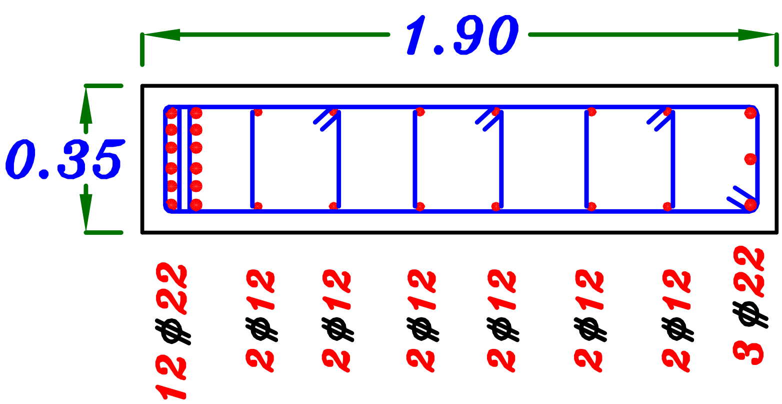
Sec. (2–2)



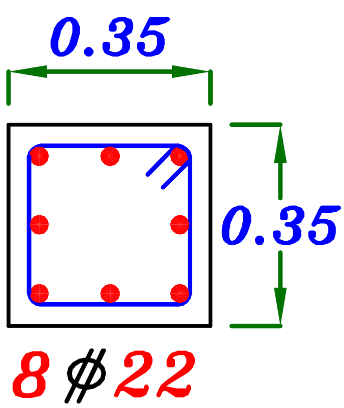
Sec. (3–3)



Sec. (4–4)

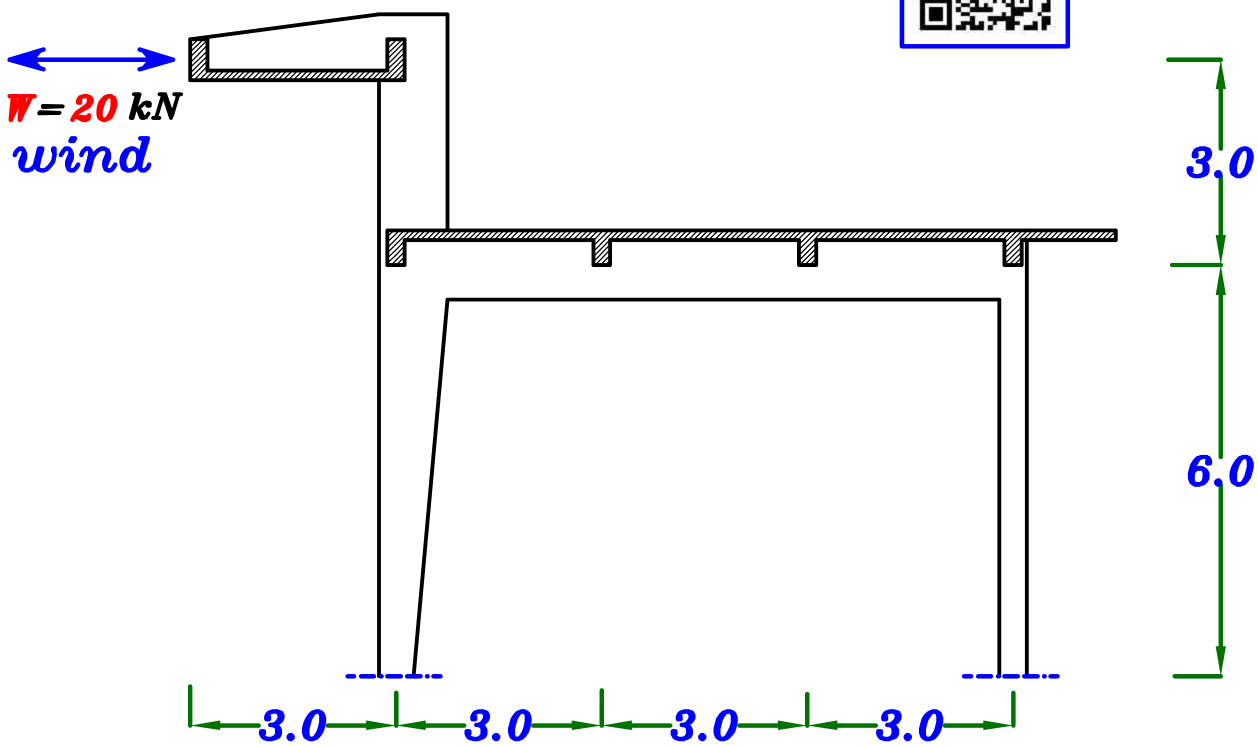


Sec. (5–5)



Sec. (6–6)

Example.



Data.

$F_{cu} = 25 \quad N\backslash mm^2$

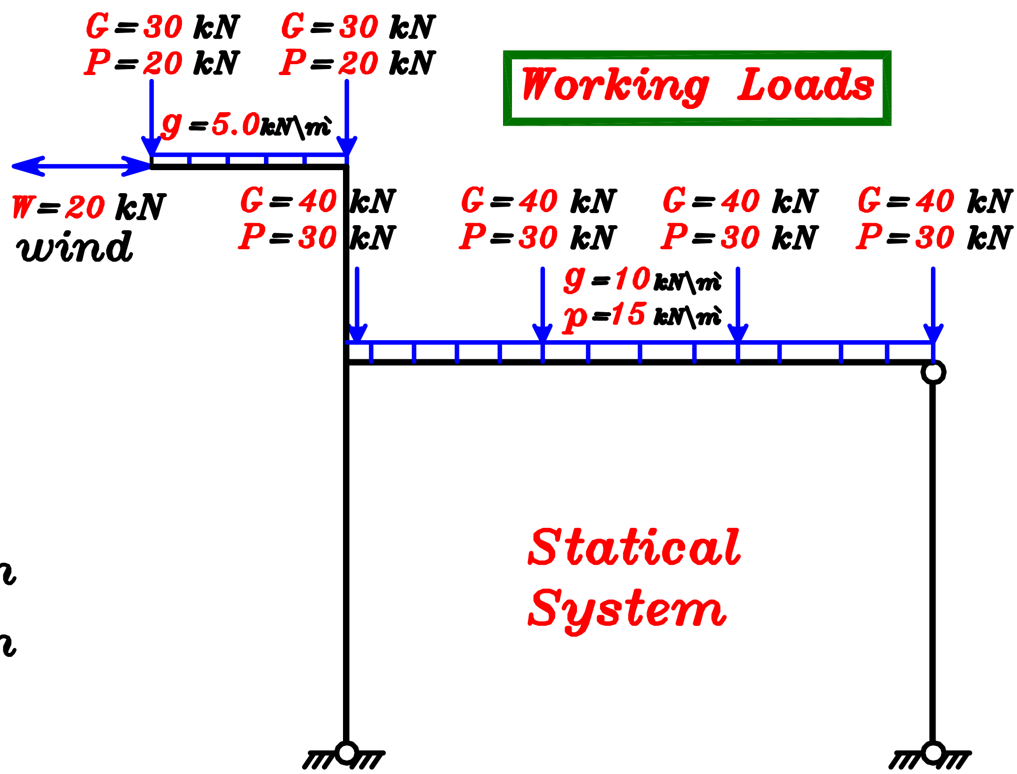
$F_y = 360 \quad N\backslash mm^2$

$t_s = 120 \quad mm$

Spacing $= 6.0 \quad m$

b (Beams) $= 250 \quad mm$

b (Frames) $= 350 \quad mm$



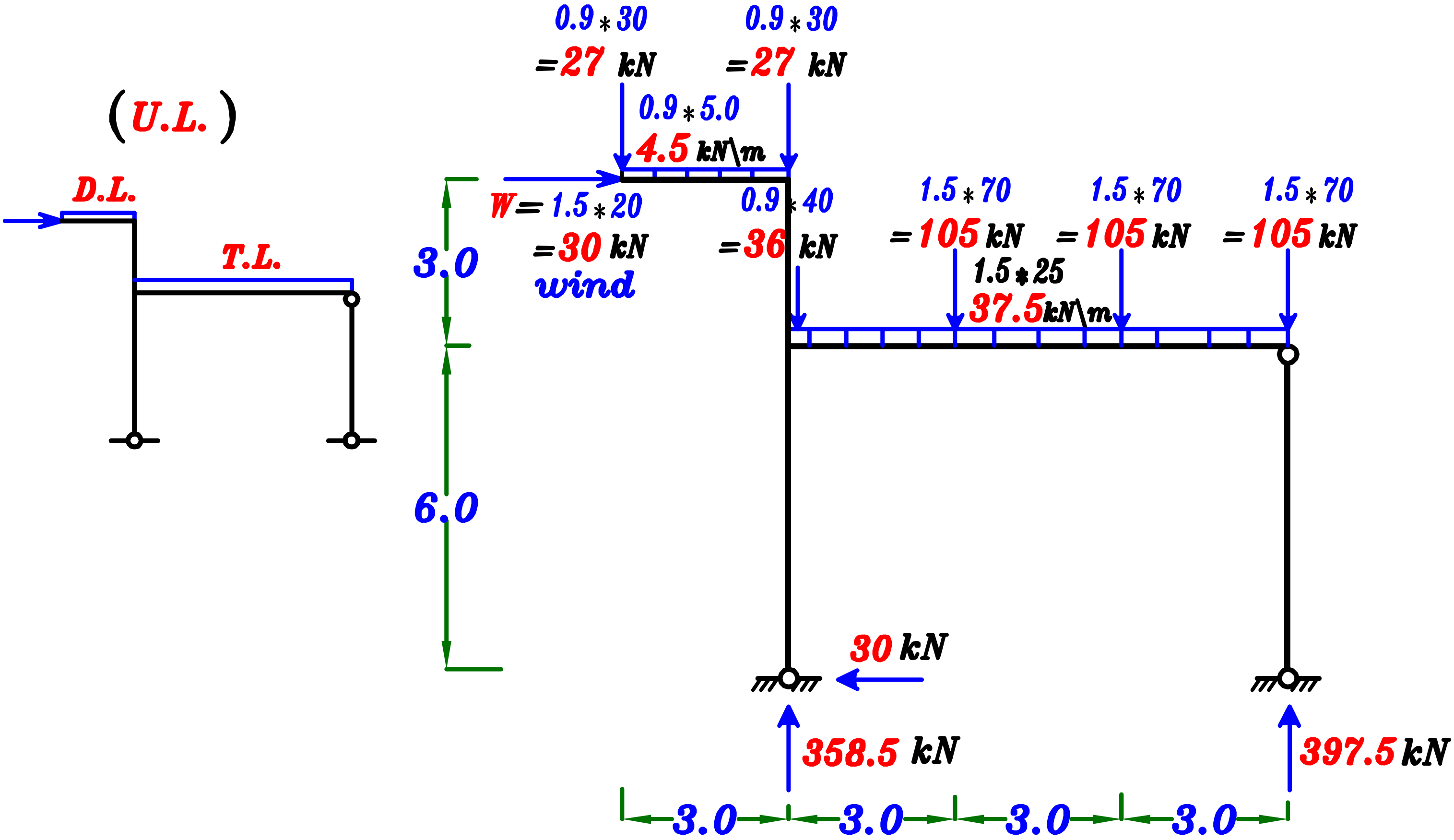
Req.

1. Draw max–max U.L. B.M.D.
2. Design the intermediate Frame & Draw Details of RFT. to scale (1:50) and cross sec. to scale (1:20)

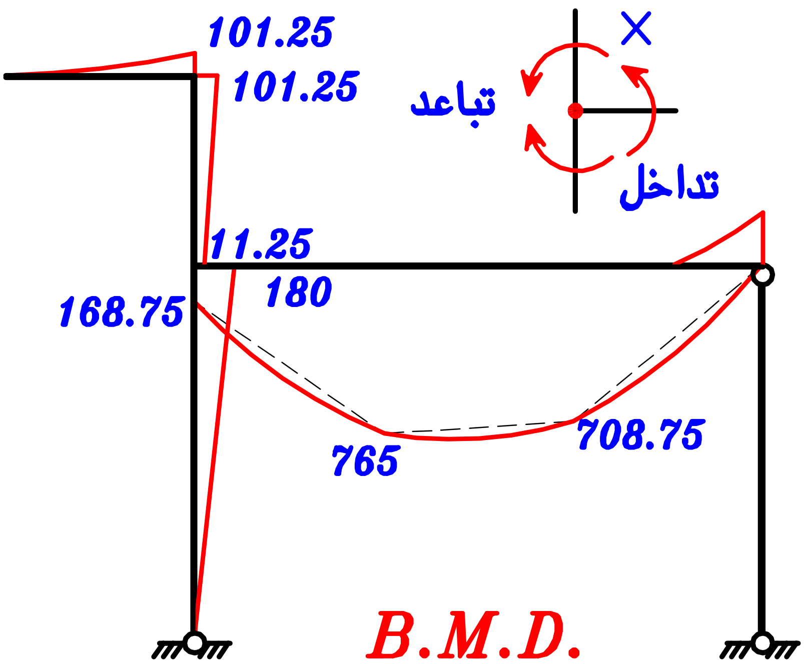
(Curtailment of steel RFT. is to be made by moment of Resistance Method)

I.F.D s

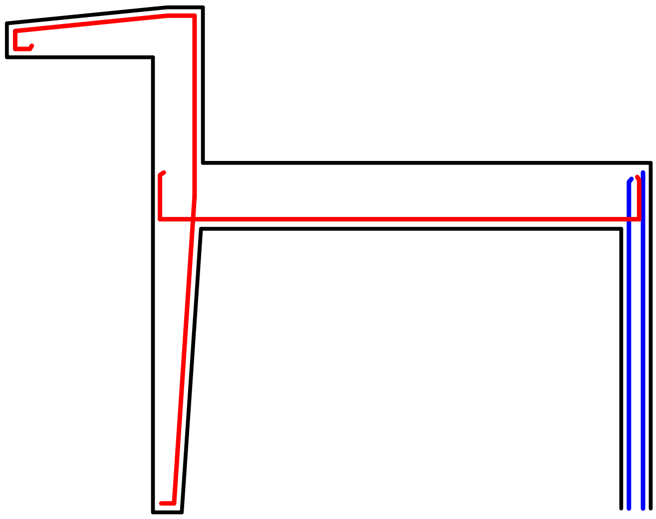
(U.L.)

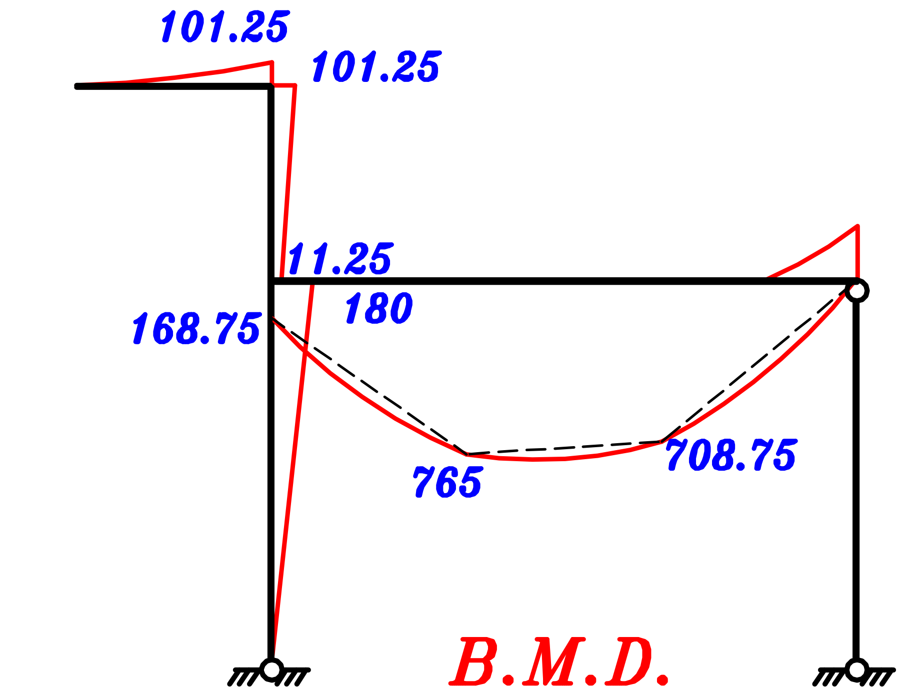


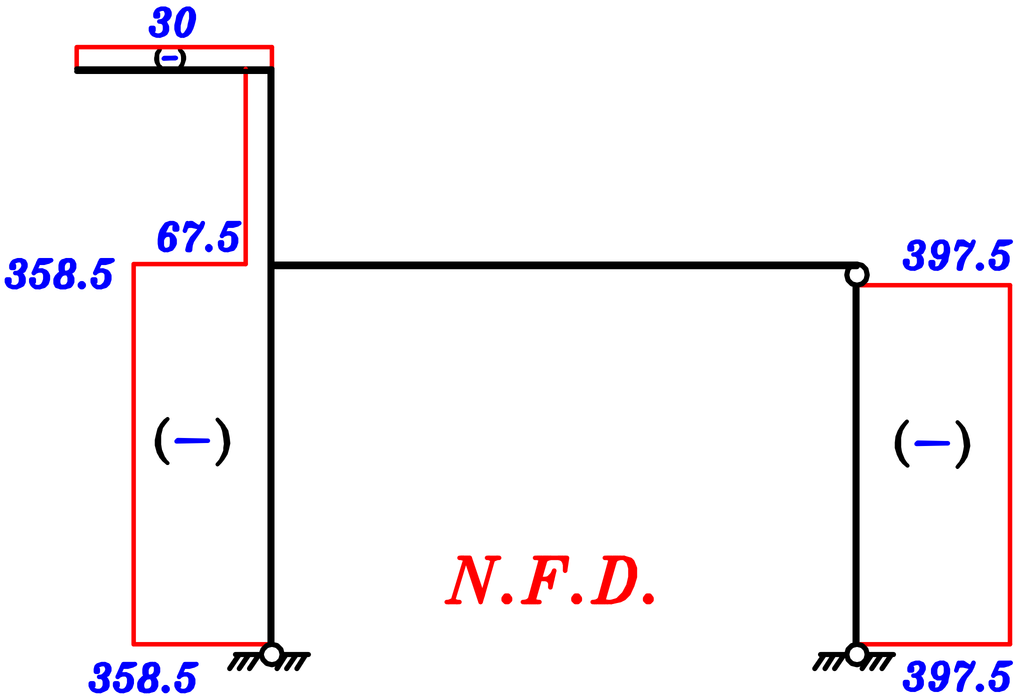
الاحمال المركزه فوق الاعمده لا يكون لها أى تأثير على قيم الـ **B.M.** و الـ **S.F.**
و لكن يكون لها تأثير على قيم الـ **N.F.** على الاعمده و للتصميم يفضل للاعمد التى عليها **B.M. & N.F.**
أن نضع فوقها **D.L.** حيث كلما قلت قيمه الـ **N.F.** ز ادت كميه حديد التسليح
أما الاعمد التى عليها **N.F.** فقط مثل **Link member** فنضع فوقها **T.L.** لزياده كميه حديد التسليح

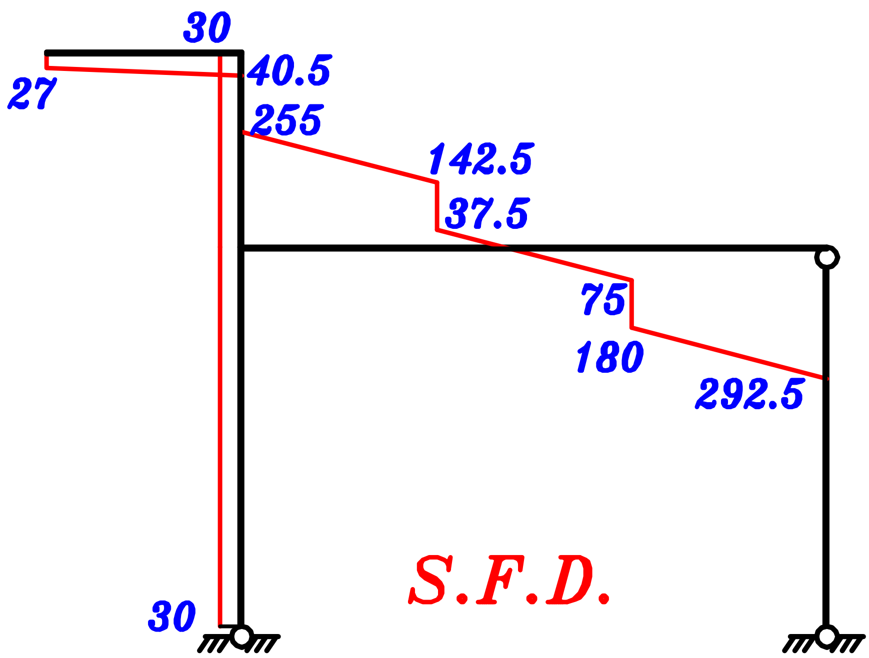


B.M.D.



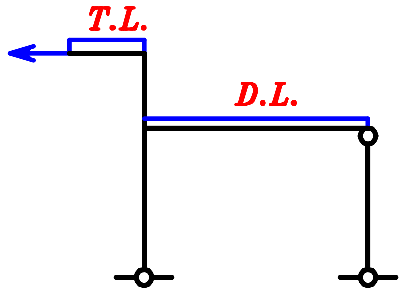

101.25
101.25
11.25
180
168.75
765
708.75
B.M.D.

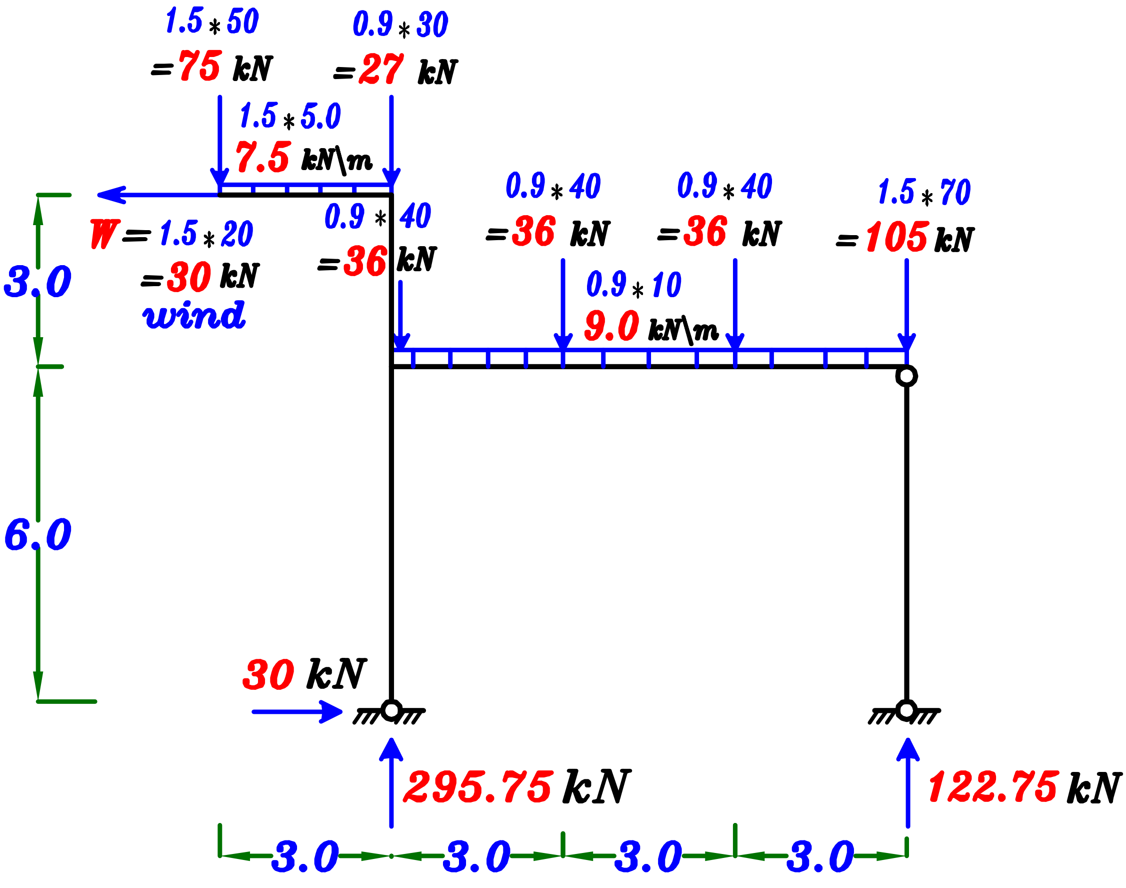

30
(−)
67.5
358.5
397.5
(−)
(−)
N.F.D.
358.5
397.5


30
27
40.5
255
142.5
37.5
75
180
292.5
S.F.D.
30

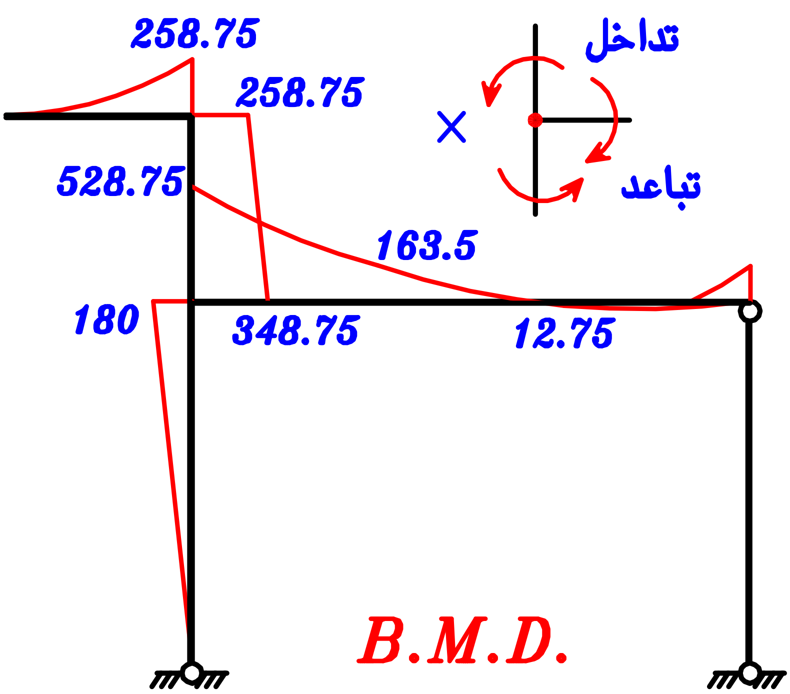
I.F.D s

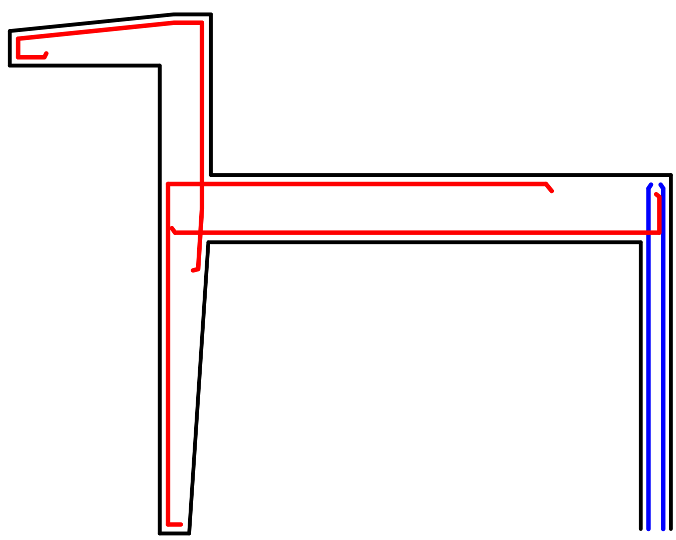
(U.L.)

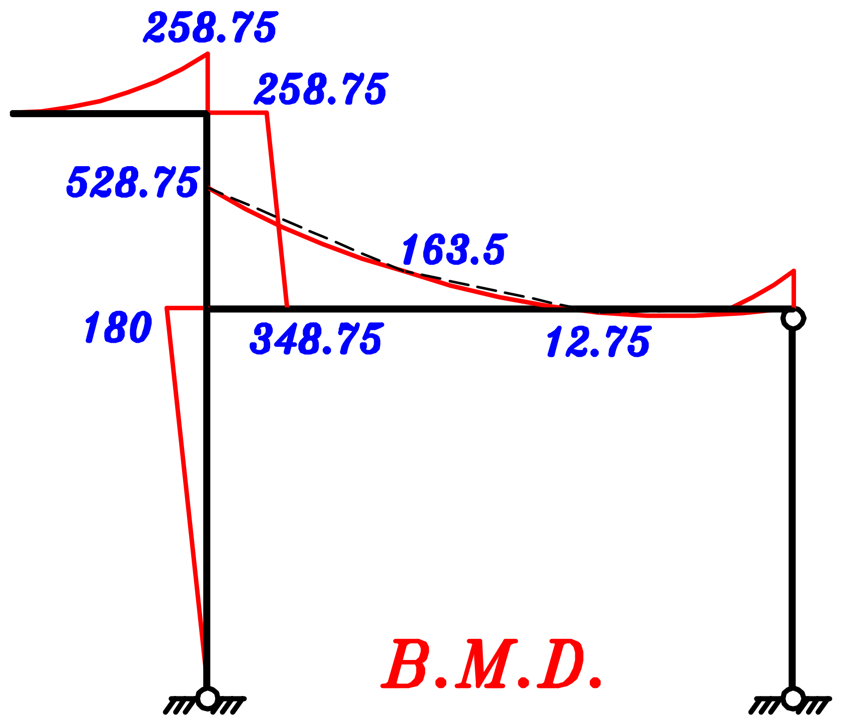


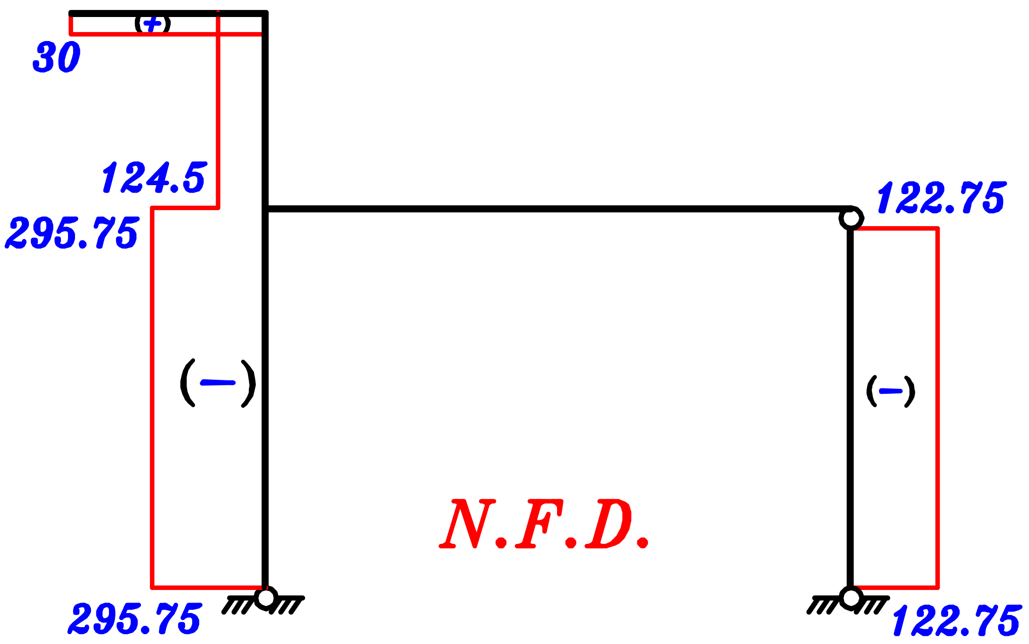


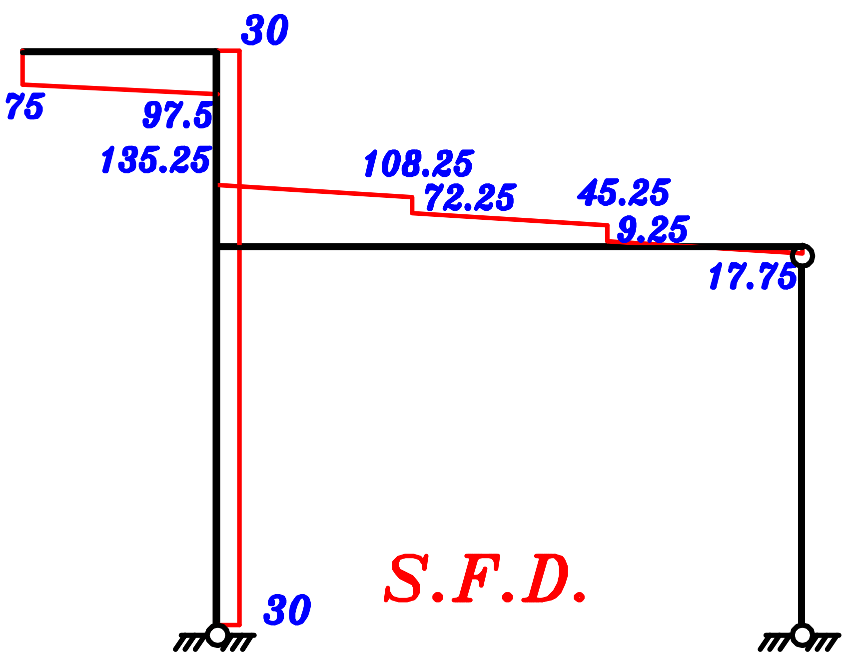
الاحمال المركزه فوق الاعمده لا يكون لها أى تأثير على قيم الـ *B.M.* و الـ *S.F.*
و لكن يكون لها تأثير على قيم الـ *N.F.* على الاعمده و للتصميم يفضل للاعمد التى عليها *N.F. & B.M.*
أن نضع فوقها *D.L.* حيث كلما قلت قيمه الـ *N.F.* ز ادت كميه حديد التسليح
أما الاعمد التى عليها *N.F.* فقط مثل *Link member* فنضع فوقها *T.L.* لزياده كميه حديد التسليح



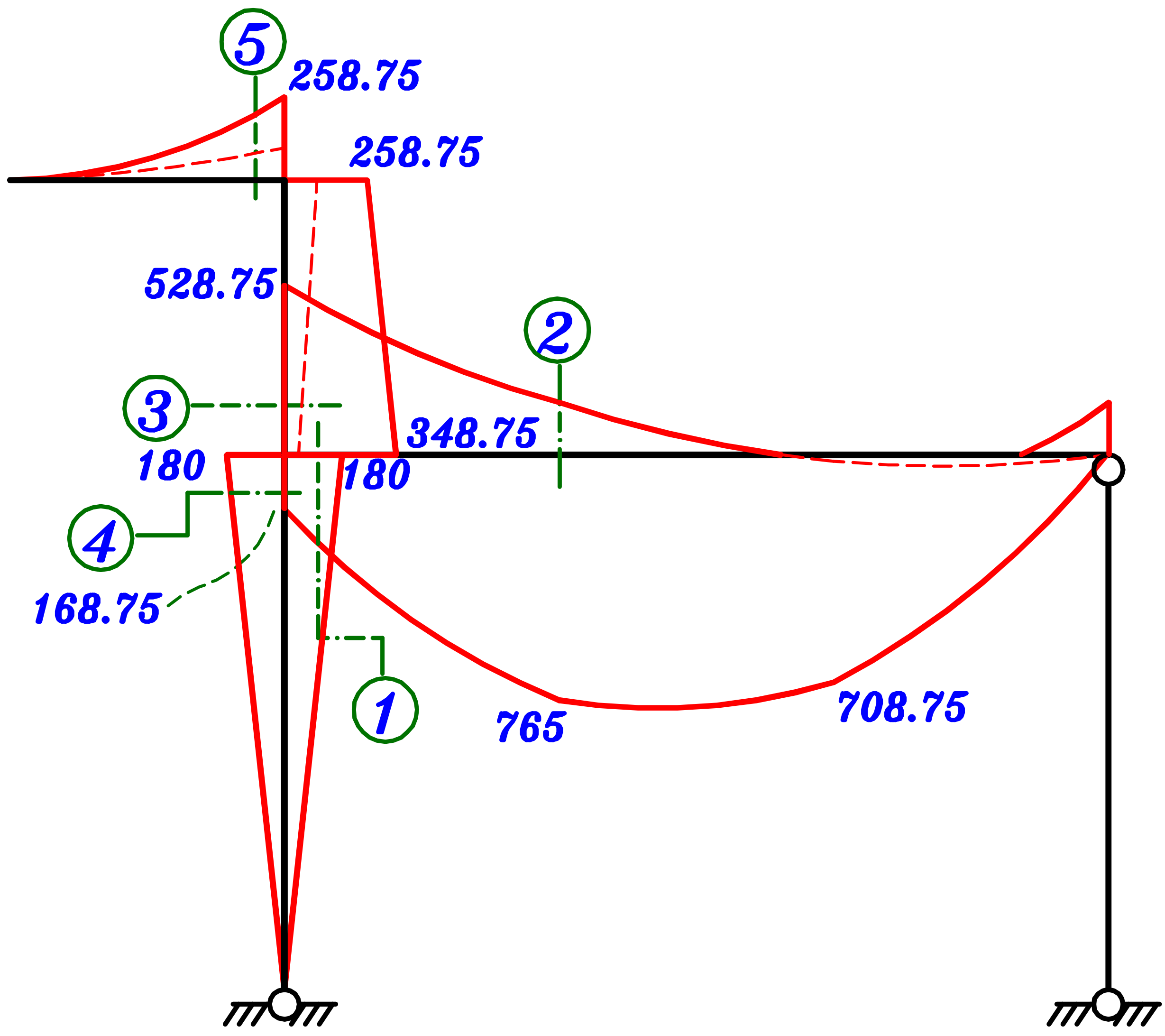



258.75
258.75
528.75
163.5
180
348.75
12.75
B.M.D.


(+)
30
124.5
295.75
122.75
(–)
(–)
N.F.D.
295.75
122.75


30
75
97.5
135.25
108.25
72.25
45.25
9.25
17.75
30
S.F.D.

(max−max) B.M.D. (U.L.)

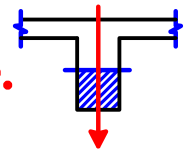


نرسم max−max B.M.D. لتحديد ال critical sections
و عند تصميم أى قطاع نحدد العزم المصمم عليه من أى حاله تحميل و نأخذ قيمه ال Normal
عليه من نفس حاله التحميل و ليس شرط أن يكون ال Normal الاكبر

لا نرسم التسليح على شكل ال max−max B.M.D.
و لكن نرسم التسليح لاى حاله تحميل أولا ثم نكمل التسليح من حاله التحميل الاخرى

Design of Sections.

Sec. ① $M = 528.75$ kN.m , $b = 350$ mm R−Sec.



Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$- \text{Get } d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} = 3.50 \sqrt{\frac{528.75 * 10^6}{25 * 350}} = 860.37 \text{ mm}$$

− Take $\boxed{d = 900 \text{ mm}}$, $\boxed{t = 950 \text{ mm}}$

$$- \text{Get } A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{528.75 * 10^6}{0.78 * 360 * 860.37} = 2188.6 \text{ mm}^2$$

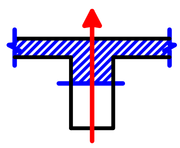
Check $A_{s_{min.}}$ $A_{s_{req.}} = 2188.6 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 900 = 984.3 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.} b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2188.6 \text{ mm}^2$ $7 \phi 20$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{20 + 25} = 7.22 = 7.0 \text{ bars}$$

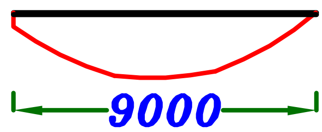
Sec. ② $M_{U.L.} = 765$ kN.m T−Sec.



Take $d = 900$ mm (The same d of Sec. ①)

$$B = \begin{cases} C.L. - C.L. = \text{Spacing} = 6.0\text{m} = 6000 \text{ mm} \\ 16\, t_s + b = 16 * 120 + 350 = 2270 \text{ mm} \\ K \dfrac{L}{5} + b = 1.0 * \dfrac{9000}{5} + 350 = 2150 \text{ mm} \end{cases}$$

$K = 1.0$



$\boxed{B = 2150 \text{ mm}}$

$\because\ d = C_1 \sqrt{\dfrac{M_{U.L.}}{F_{cu} B}} \quad \therefore\ 900 = C_1 \sqrt{\dfrac{765 * 10^6}{25 * 2150}} \longrightarrow C_1 = 7.54 \longrightarrow J = 0.826$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{765 * 10^6}{0.826 * 360 * 900} = 2858.5\ mm^2$$

Check $A_{s_{min.}}$

$A_{s_{req.}} = 2858.5\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 900 = 984.3\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2858.5\ mm^2$ $\quad 10\phi 20$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 2858.5 \quad 4\phi 12$

Sec. ③

$M = 348.75\ kN.m\ ,\ P = 124.5\ kN\ ,\ b = 350\ mm$

Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$d_o = 3.5 \sqrt{\frac{348.75 * 10^6}{25 * 350}} = 698.75\ mm \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(698.75) = (768.6 \rightarrow 908.3)\ mm$$

Take $d = 900\ mm\ ,\ t = 900 + 50 = 950\ mm$

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{124.5 * 10^3}{25 * 350 * 900} = 0.015 < 0.04 \quad \therefore$ (neglect P)

$\therefore d = d_o = 698.75\ mm \quad \boxed{d = 700\ mm}\ ,\ \boxed{t = 750\ mm}$

$\because\ \boxed{t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)} = 950\ mm}$

$$\because d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 900 = C_1\sqrt{\frac{348.75 * 10^6}{25 * 350}} \rightarrow C_1 = 4.50 \rightarrow J = 0.819$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{348.75 * 10^6}{0.819 * 360 * 900} = 1314.2 \ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1314.2 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 900 = 984.3 \ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1314.2 \ mm^2 \qquad 5\phi 20$$

Sec. ④ $\quad M = 180 \ kN.m \ , \ P = 295.75 \ kN \ , \ b = 350 \ mm$

$d = 900 \ mm$ (the same depth of Sec. ③)

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{295.75 * 10^3}{25 * 350 * 950} = 0.035 < 0.04 \quad \therefore (\text{neglect } P)$$

$$\because d = C_1\sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 900 = C_1\sqrt{\frac{180 * 10^6}{25 * 350}} \rightarrow C_1 = 6.27 \rightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{180 * 10^6}{0.826 * 360 * 900} = 672.59 \ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s\ req.} = 672.6 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 900 = 984.27 \ mm^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$A_{s_{min.}} = 0.225 * \frac{\sqrt{F_{cu}}}{F_y}\, b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 900 = 984.27$$

$$1.3\ A_{s_{req.}} = 1.3 * 672.6 = 874.37$$

الأقل $= 874.37$

st. 360/520 $\qquad \frac{0.15}{100}\, b\, d = \frac{0.15}{100} * 350 * 900 = 472.5 \ mm^2$

الأكبر $= 874.37 \ mm^2$

$3\phi 20$

Sec. ⑤ $M = 258.75$ kN.m , $T = 30.0$ kN , $b = 350$ mm

Take $C_1 = 3.50 \longrightarrow J = 0.78$

$$d_o = 3.5\sqrt{\frac{258.75 * 10^6}{25 * 350}} = 601.8 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (0.9 \rightarrow 1.0)\, d_o = (0.9 \rightarrow 1.0)\,(601.8) = (541.7 \rightarrow 601.8) \text{ mm}$$

Take $\boxed{d = 550 \text{ mm}}$, $\boxed{t = 550 + 50 = 600 \text{ mm}}$

$$e = \frac{M}{T} = \frac{258.75}{30.0} = 8.625 \text{ m} \quad \therefore \frac{e}{t} = \frac{8.625}{0.6} = 14.37 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e - \frac{t}{2} + c = 8.625 - \frac{0.60}{2} + 0.05 = 8.375 \text{ m}$$

$$M_s = T * e_s = 30.0 * 8.375 = 251.25 \text{ kN.m}$$

$$\therefore d = C_1\sqrt{\frac{M_s}{F_{cu}\, b}} \qquad \therefore 550 = C_1\sqrt{\frac{251.25 * 10^6}{25 * 350}} \rightarrow C_1 = 3.24 \rightarrow J = 0.764$$

$$\therefore A_s = \frac{M_s}{J\, F_y\, d} + \frac{T_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{251.25 * 10^6}{0.764 * 360 * 550} + \frac{30.0 * 10^3}{(360 \backslash 1.15)} = 1756.75 \text{ mm}^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 1756.75 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 700 = 765.6 \text{ mm}^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1756.75 \text{ mm}^2 \quad 6\phi 20$$

$$Y = \begin{cases} \dfrac{t}{2} = \dfrac{600}{2} = 300 \text{ mm} \\ t_b = \dfrac{\text{spacing}}{12} = \dfrac{6000}{12} = 500 \text{ mm} \\ t - \dfrac{L_c}{3} = 600 - \dfrac{3000}{3} = -400 \text{ mm} \end{cases} \qquad \boxed{Y = 500 \text{ mm}}$$

Sec. ⑥ (350*450) Axially Loaded Column. $P = 397.5\ kN$

$$\therefore P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$$

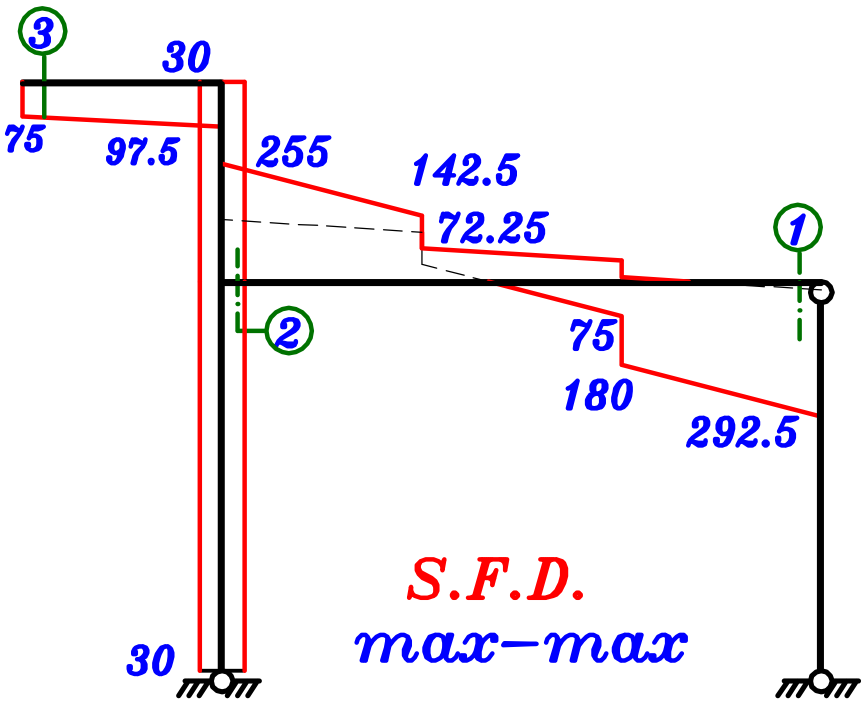
$$\therefore 397.5 * 10^3 = 0.35\,(350 * 450)(25) + 0.67\, A_s\,(360)$$

$$\therefore A_s = -\ 4065.6\ mm^2 = (-\ Ve)\ \text{Value}$$

$$\therefore A_s = A_{s_{min.}} = \frac{0.8}{100} * 350 * 450 = 1260\ mm^2$$

12 ϕ 12

Check Shear.



S.F.D.
max–max

– Allowable shear stress.

$$- \; q_{cu} = 0.24 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.24 \sqrt{\frac{25}{1.5}} = 0.98 \; N\backslash mm^2$$

$$- \; q_{max.} = 0.7 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.7 \sqrt{\frac{25}{1.5}} = 2.85 \; N\backslash mm^2$$

Sec. 1 $Q = 292.5 \; kN$

$$\therefore \text{Actual shear stress} = q_U = \frac{Q}{b\,d} = \frac{292.5 * 10^3}{350 * 900} = 0.92 \; N\backslash mm^2$$

$q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m'}$

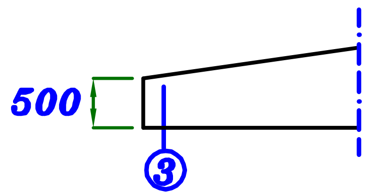
Sec. 2 $Q = 255 \; kN$

$$\therefore \text{Actual shear stress} = q_U = \frac{Q}{b\,d} = \frac{255 * 10^3}{350 * 900} = 0.81 \; N\backslash mm^2$$

$q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m'}$

Sec. 3 $Q = 75.0 \; kN$

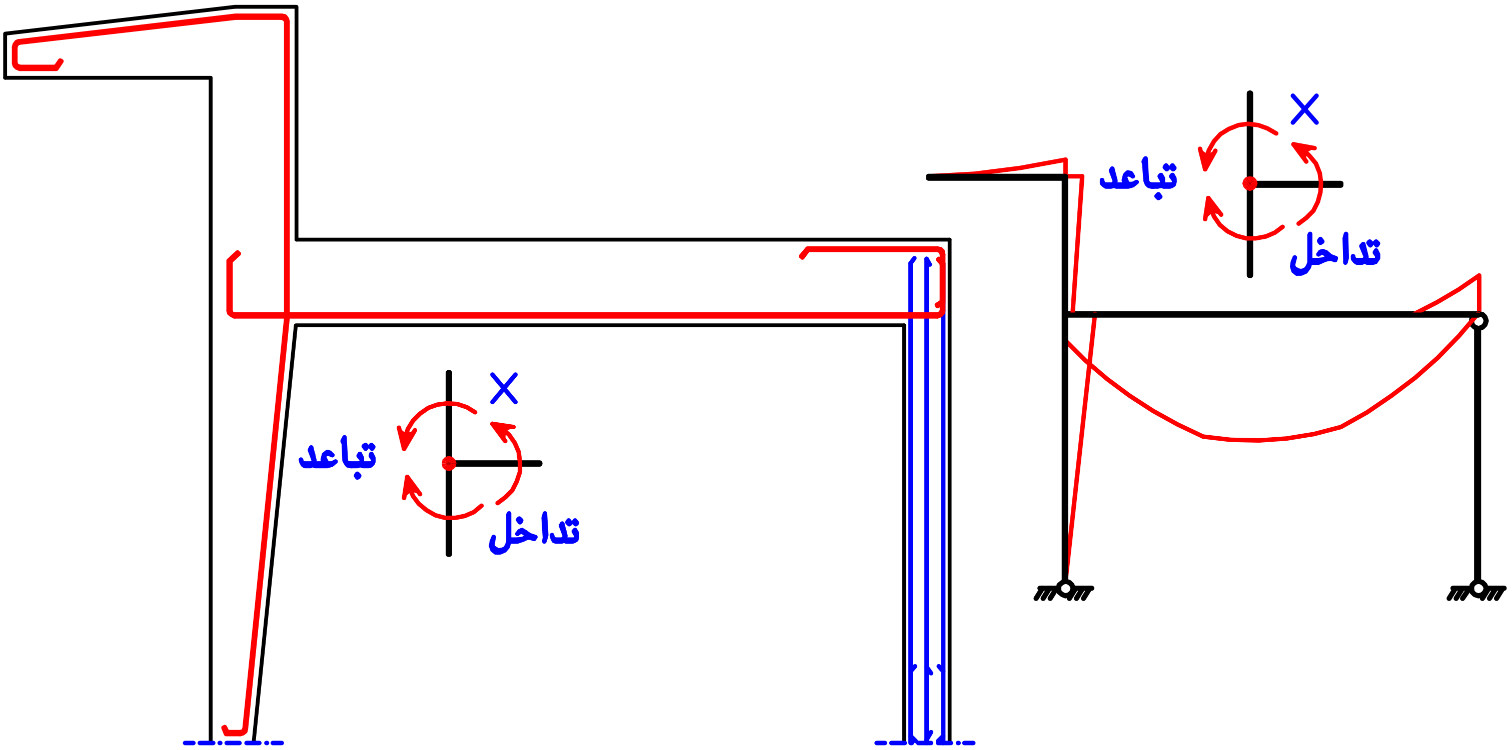
$$\therefore \text{Actual shear stress} = q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\, d^2}$$

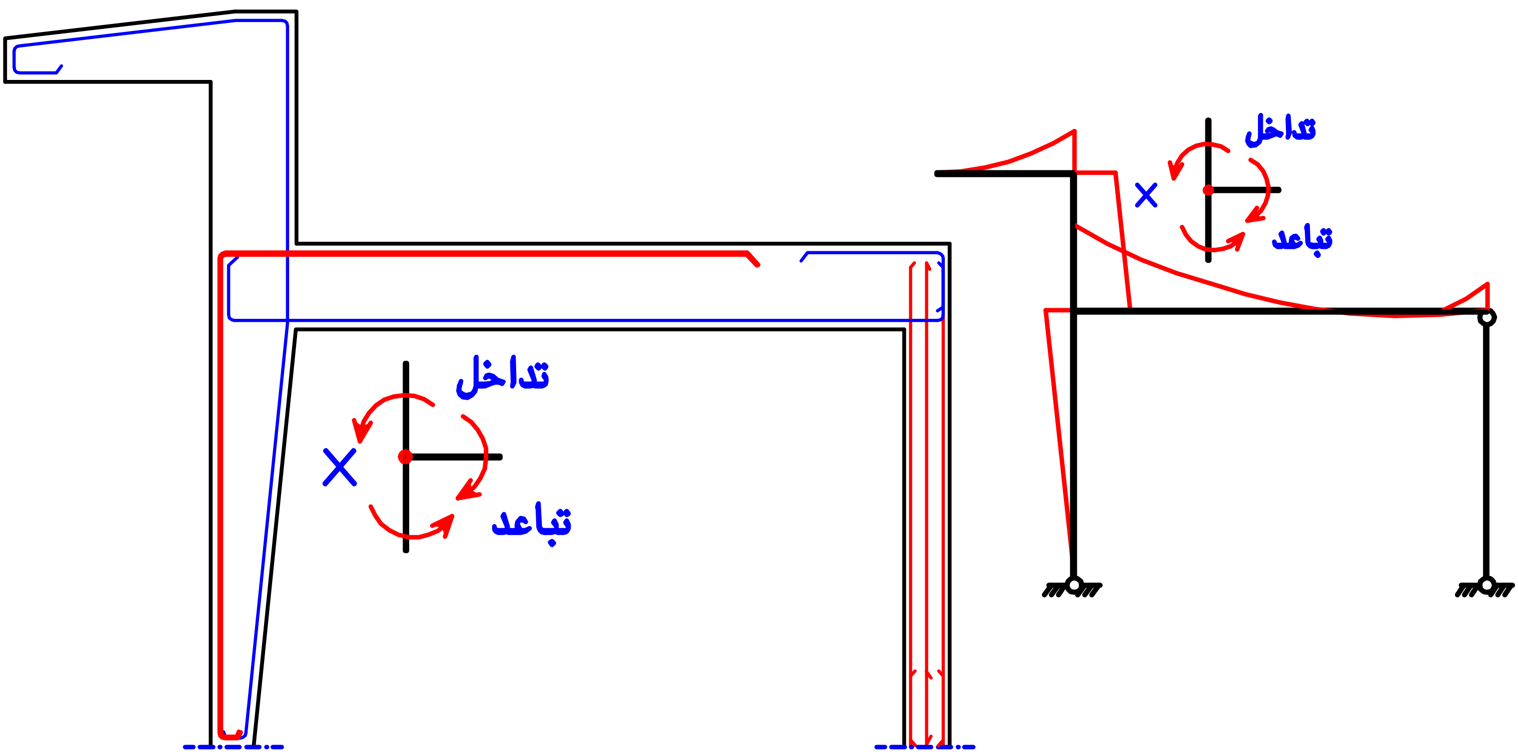


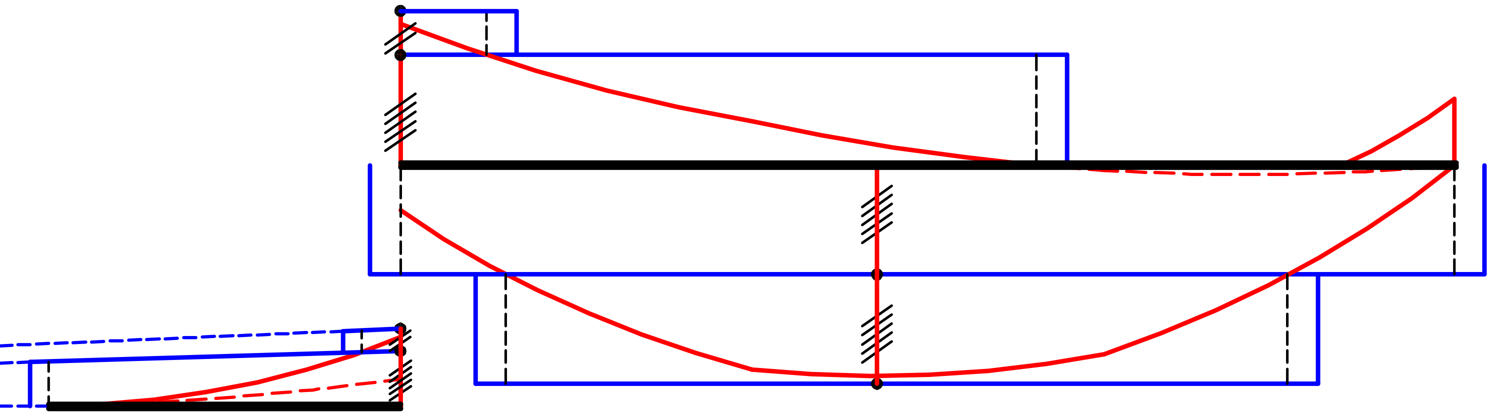
$$= \frac{75.0 * 10^3}{350 * 450} - ZERO = 0.47 \; N\backslash mm^2$$

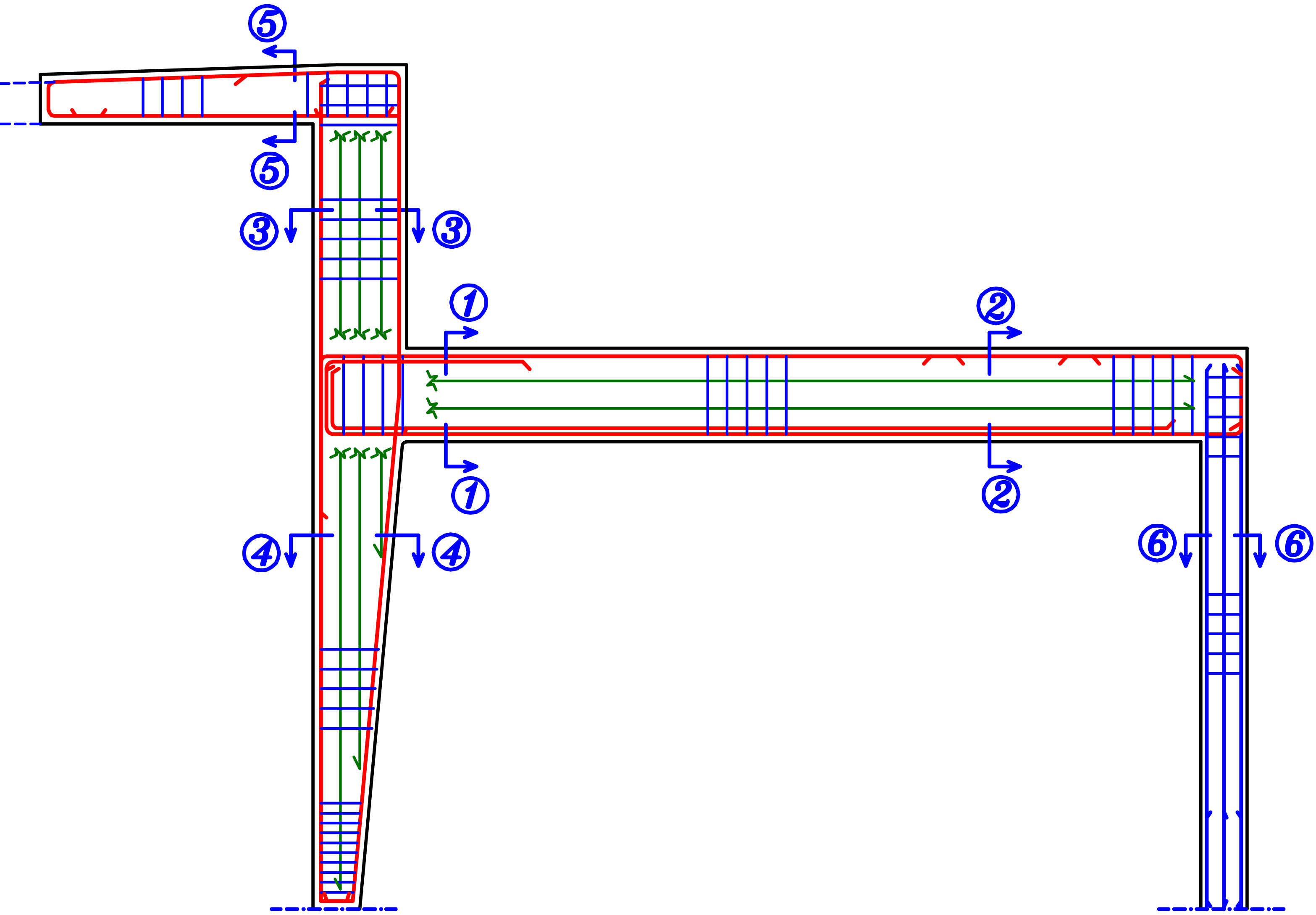
$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m'}$

لا نرسم التسليح على شكل ال *max−max* *B.M.D.*
و لكن نرسم التسليح لاى حاله تحميل أولا ثم نكمل التسليح من حاله التحميل الاخرى





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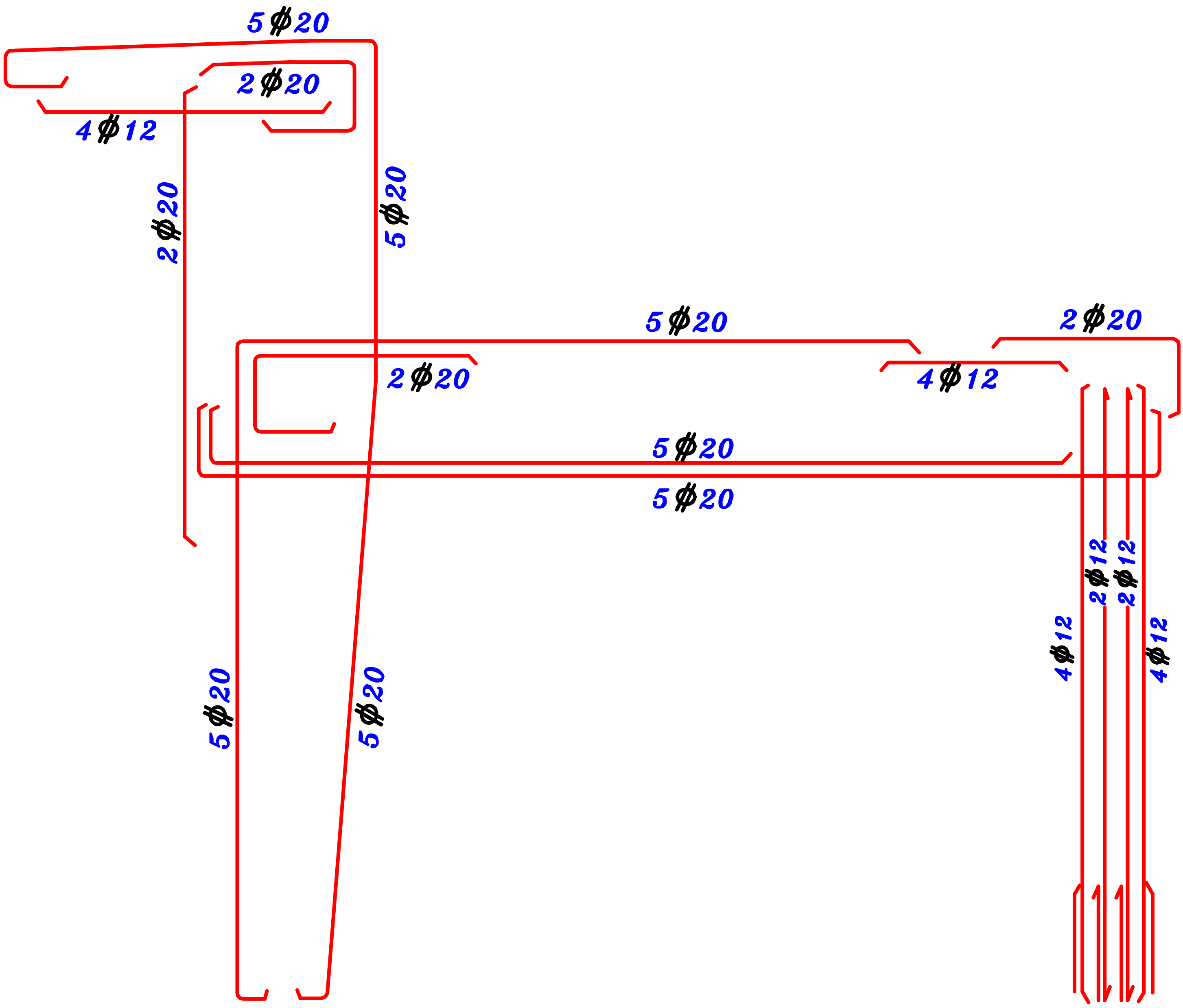
6
6


5 Ø20
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4 Ø12
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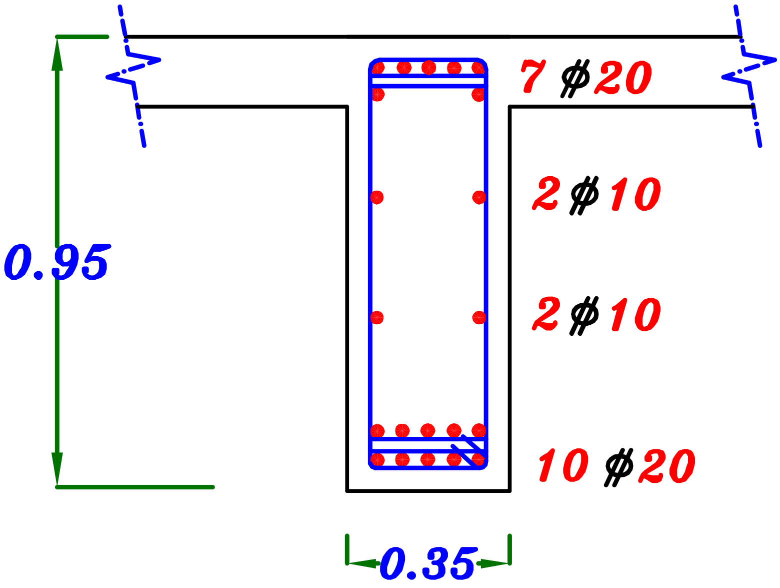
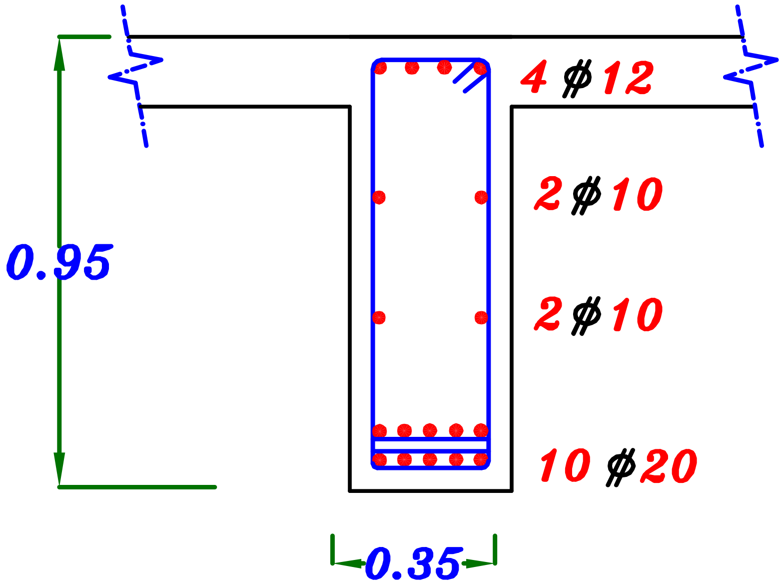
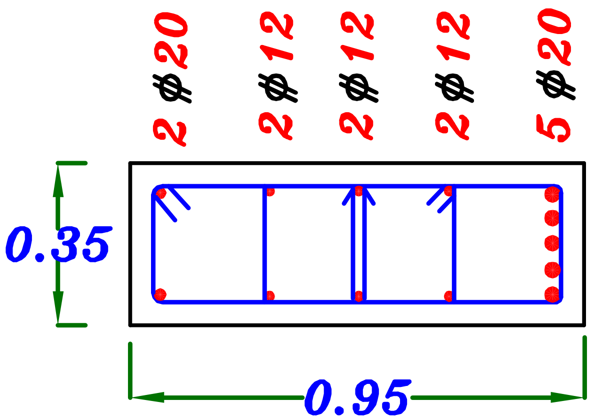
5 Ø20



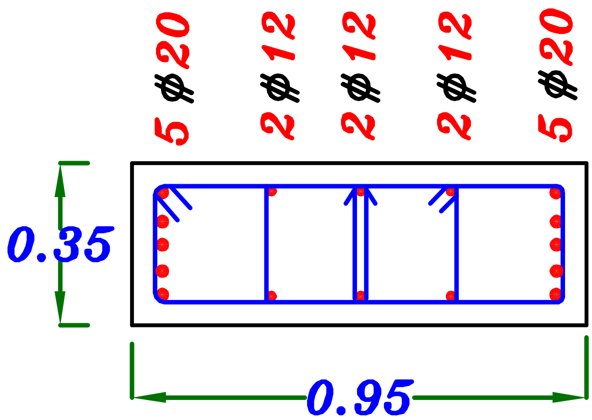
Sec. (1–1)



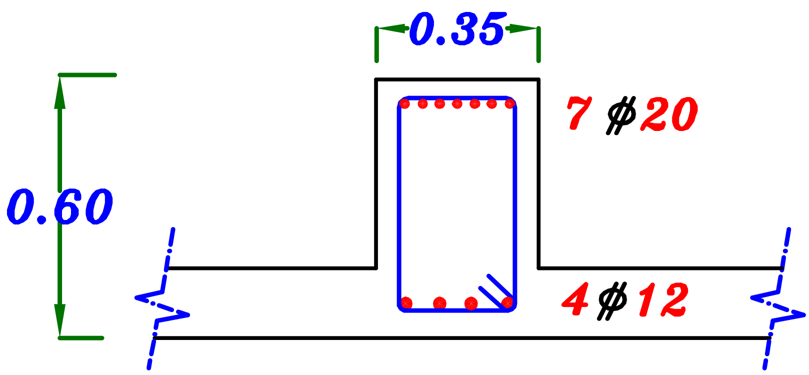
Sec. (2–2)



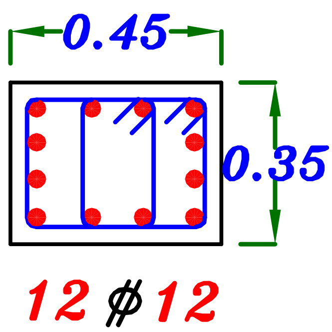
Sec. (3–3)



Sec. (4–4)

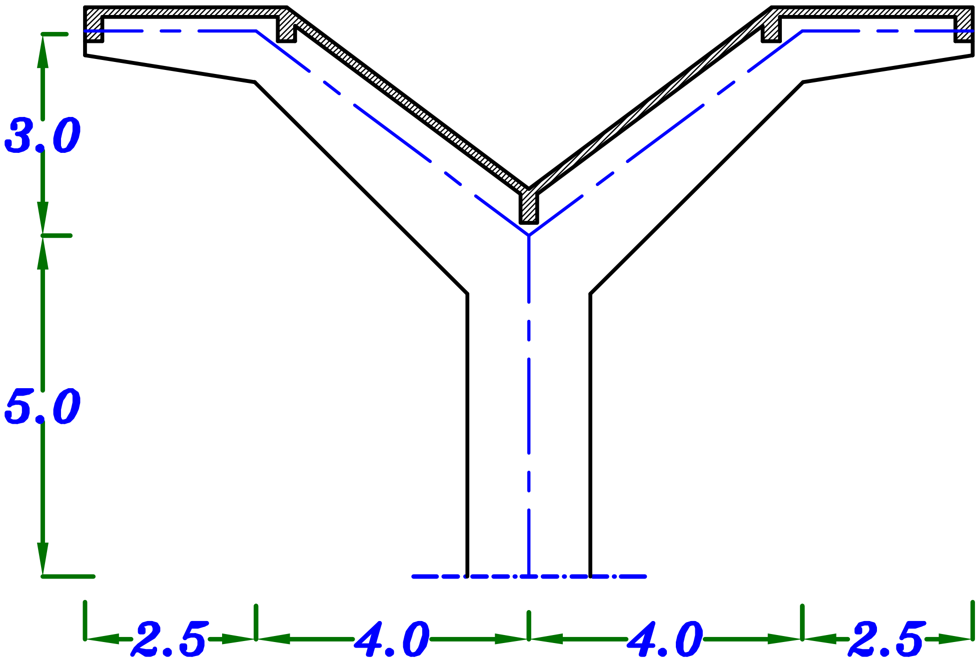


Sec. (5–5)



Sec. (6–6)

Example.



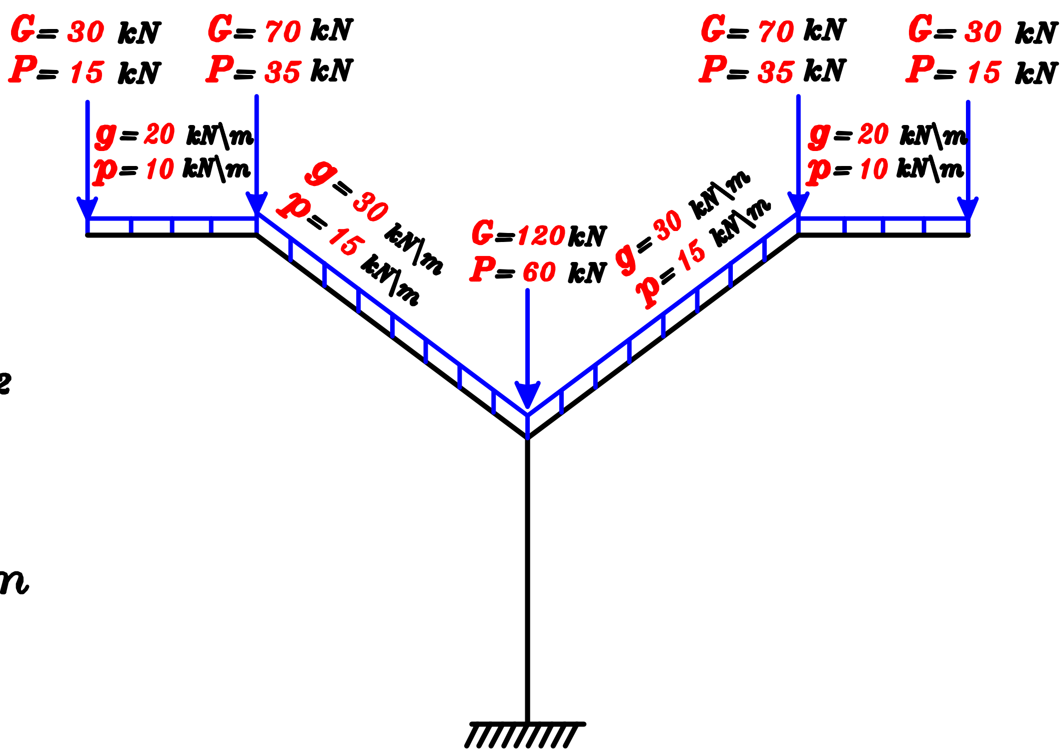
Data.

$F_{cu} = 25 \quad N\backslash mm^2$

$F_y = 360 \quad N\backslash mm^2$

$t_s = 120$ mm

Spacing = 5.0 m



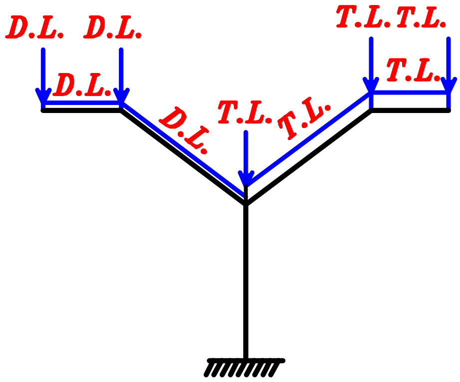
Req.

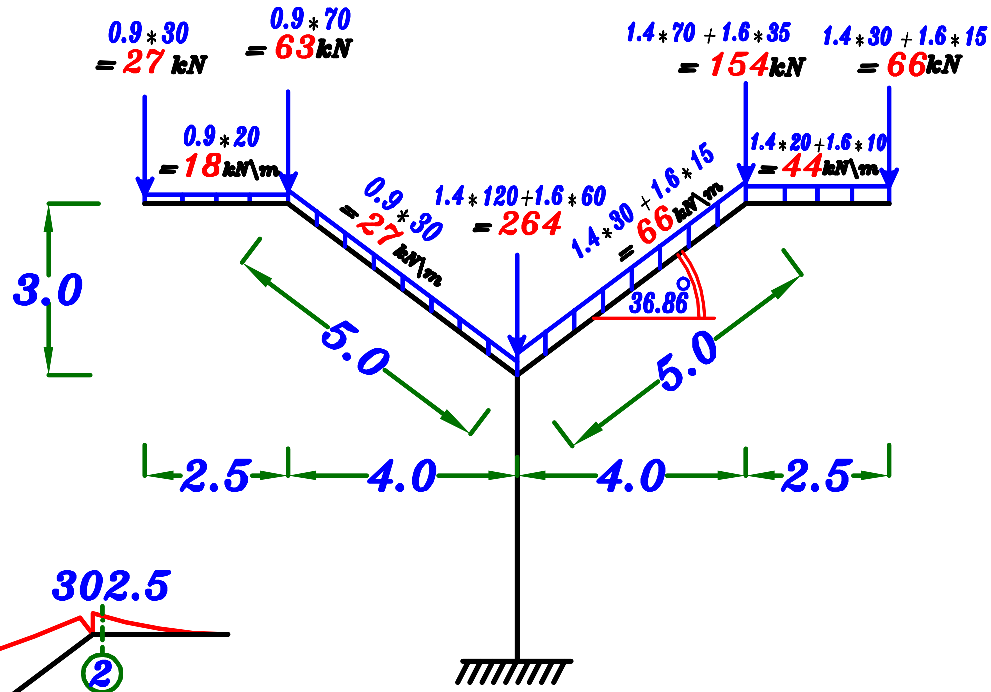
① Draw the Internal Forces Diagrams For the Frame due to the given working Loads. (max–max I.F.D.)

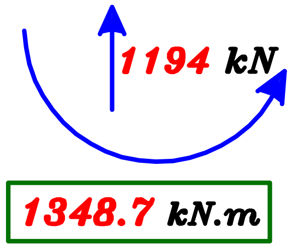
② Design the Frame.
using U.L. design method in bending.

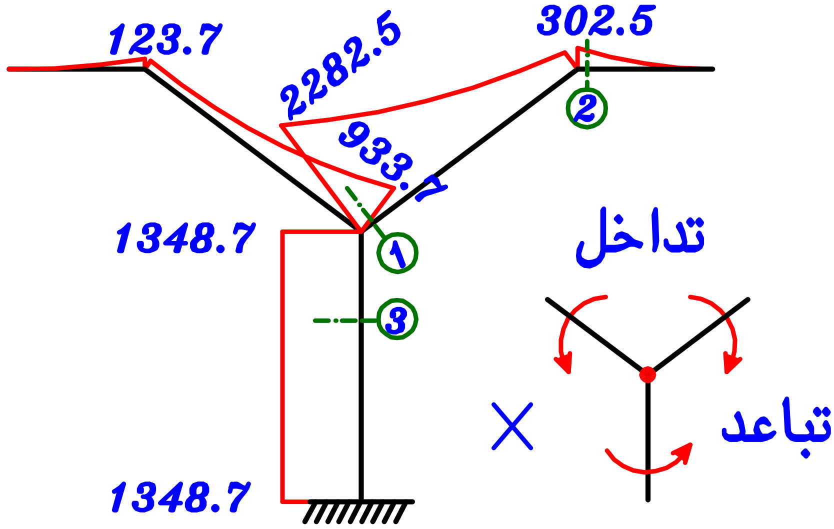
③ Draw Details of RFT. For Frame. in elevation to scale 1:50 and cross-section to scale 1:10
making curtailment of steel using Moment of Resistance Method.

1- max (-ve) B.M.D.

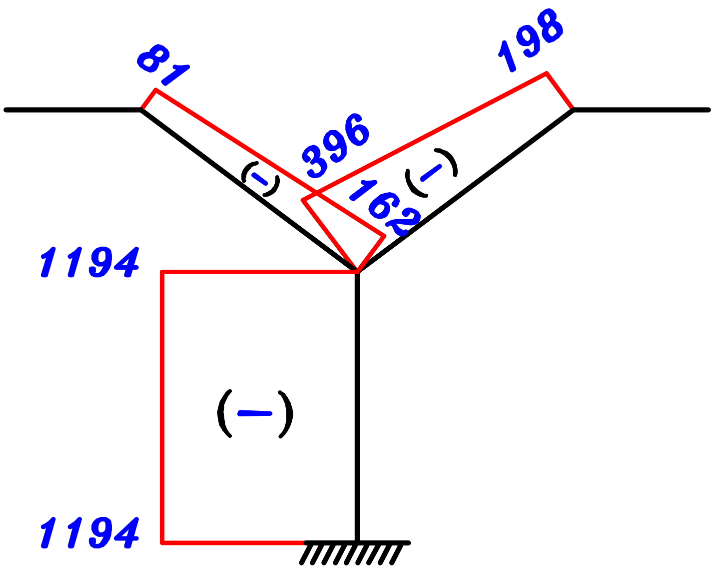




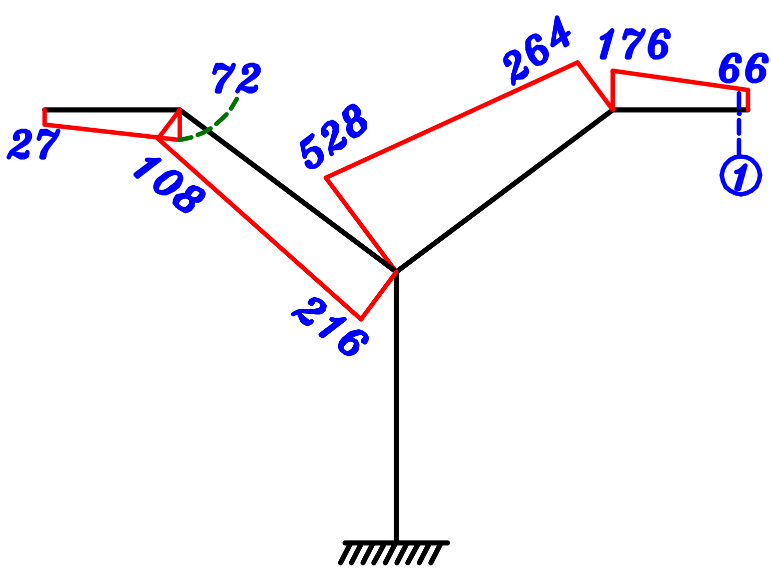




B.M.D.
(U.L.)

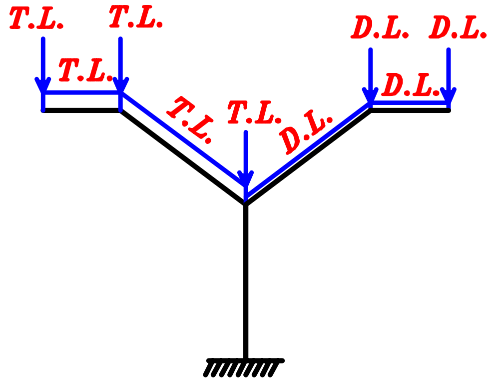


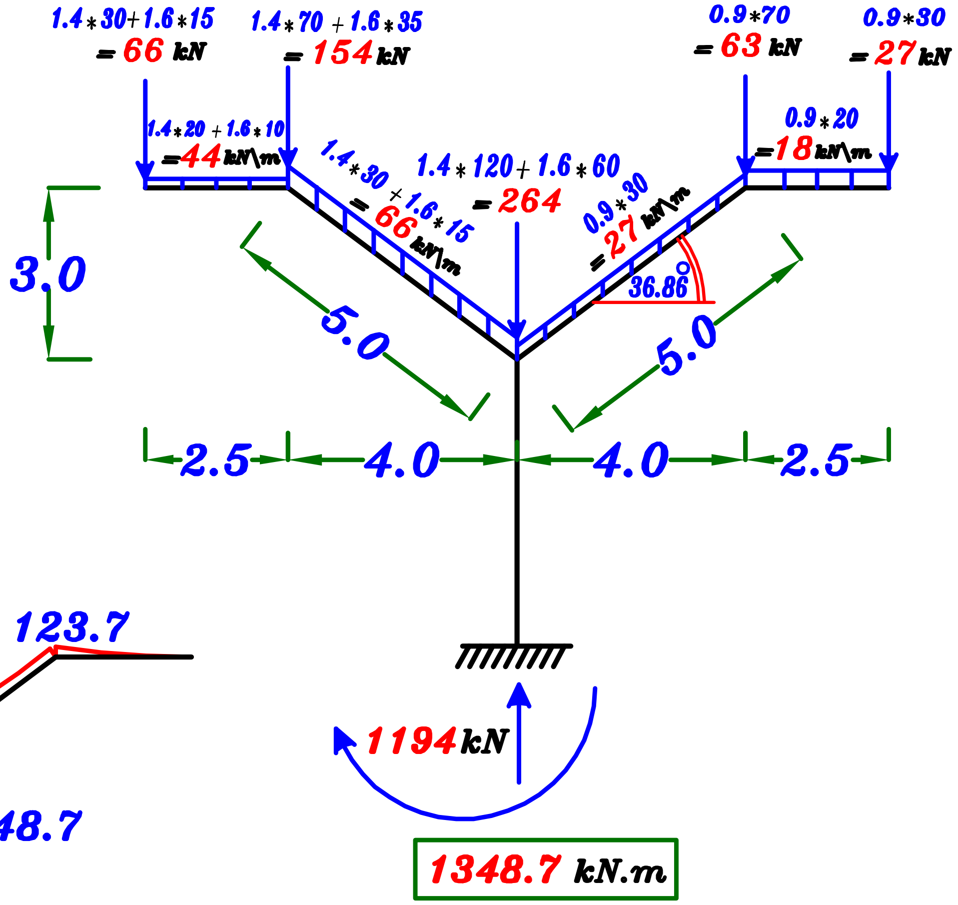
N.F.D.
(U.L.)

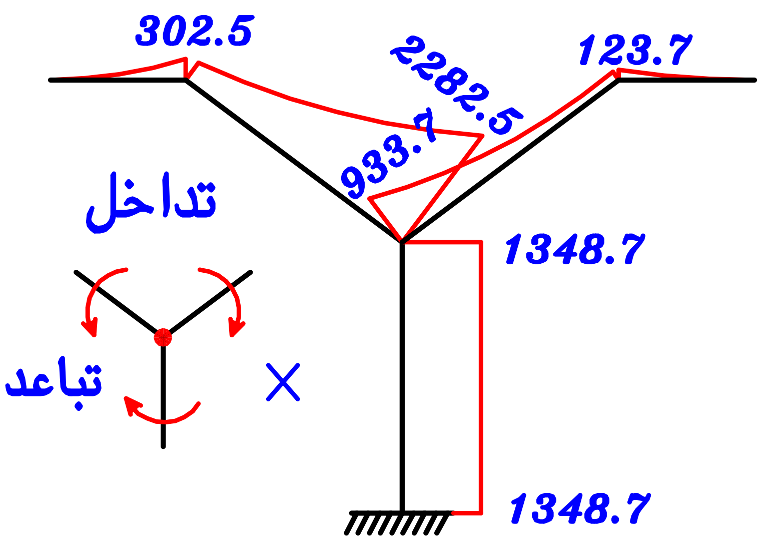


S.F.D.
(U.L.)

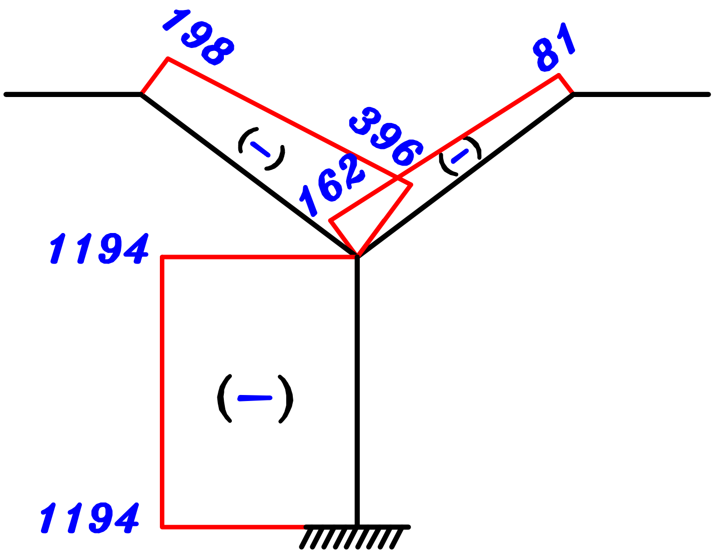
2- max (+ve) B.M.D.



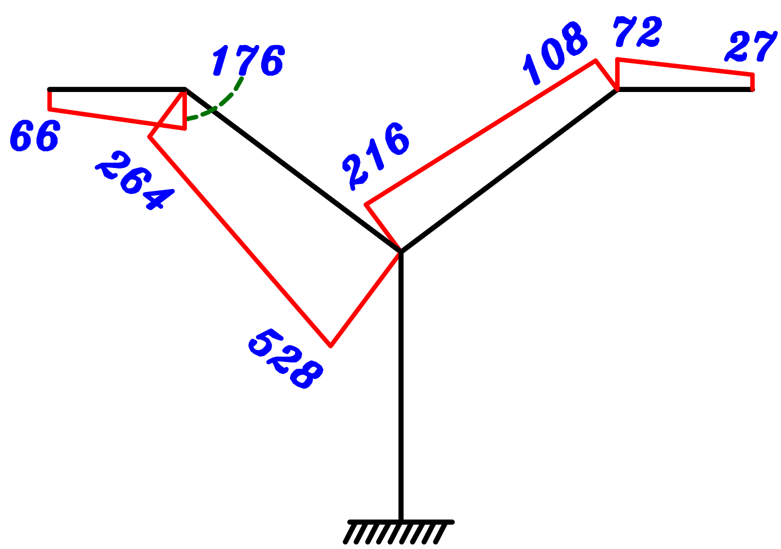




B.M.D.
(U.L.)

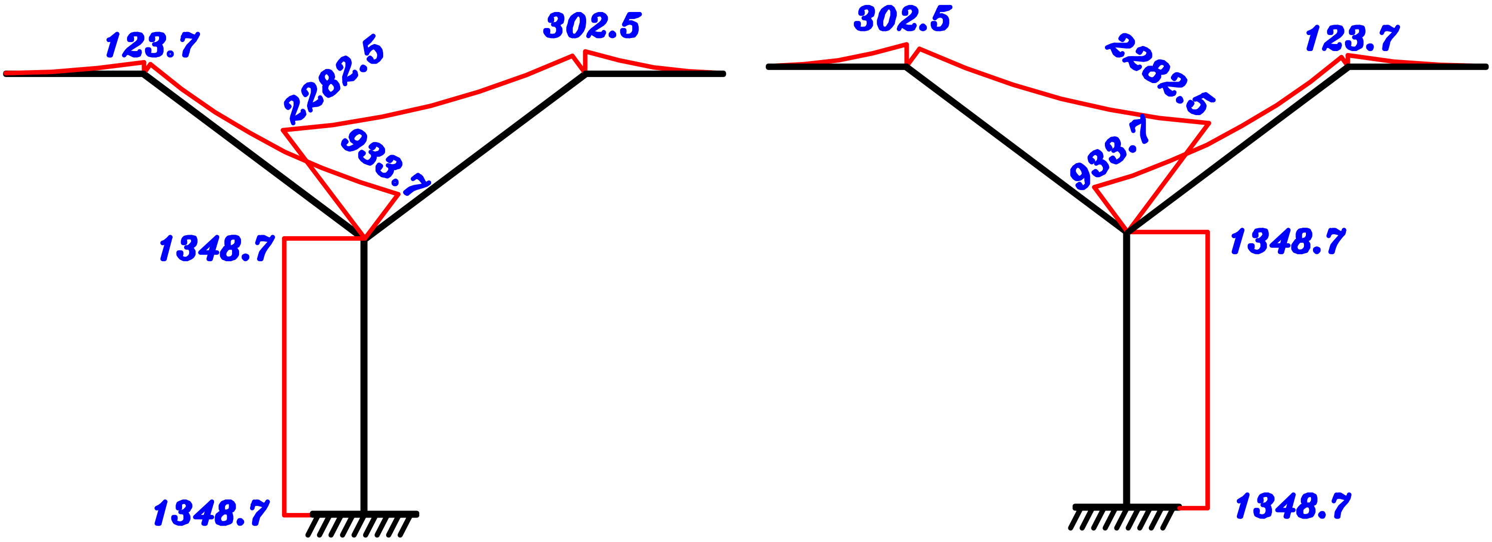


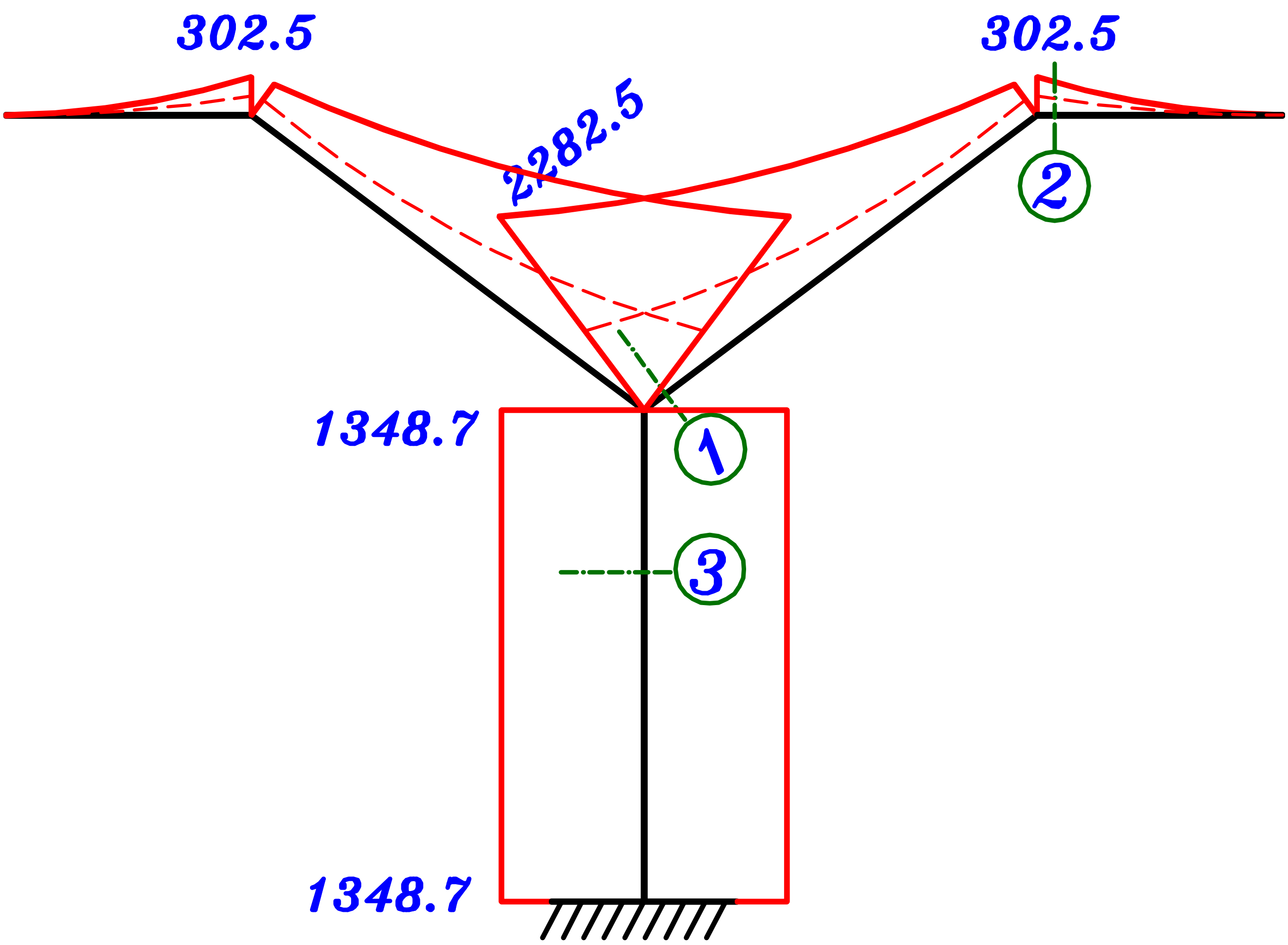
N.F.D.
(U.L.)



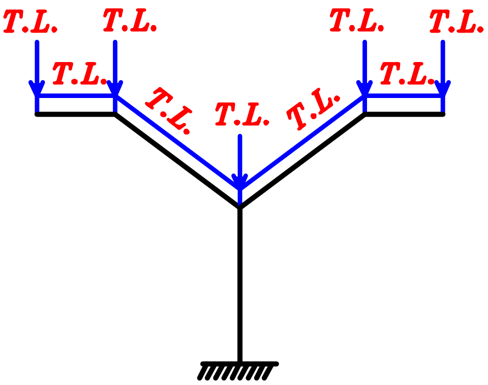
S.F.D.
(U.L.)

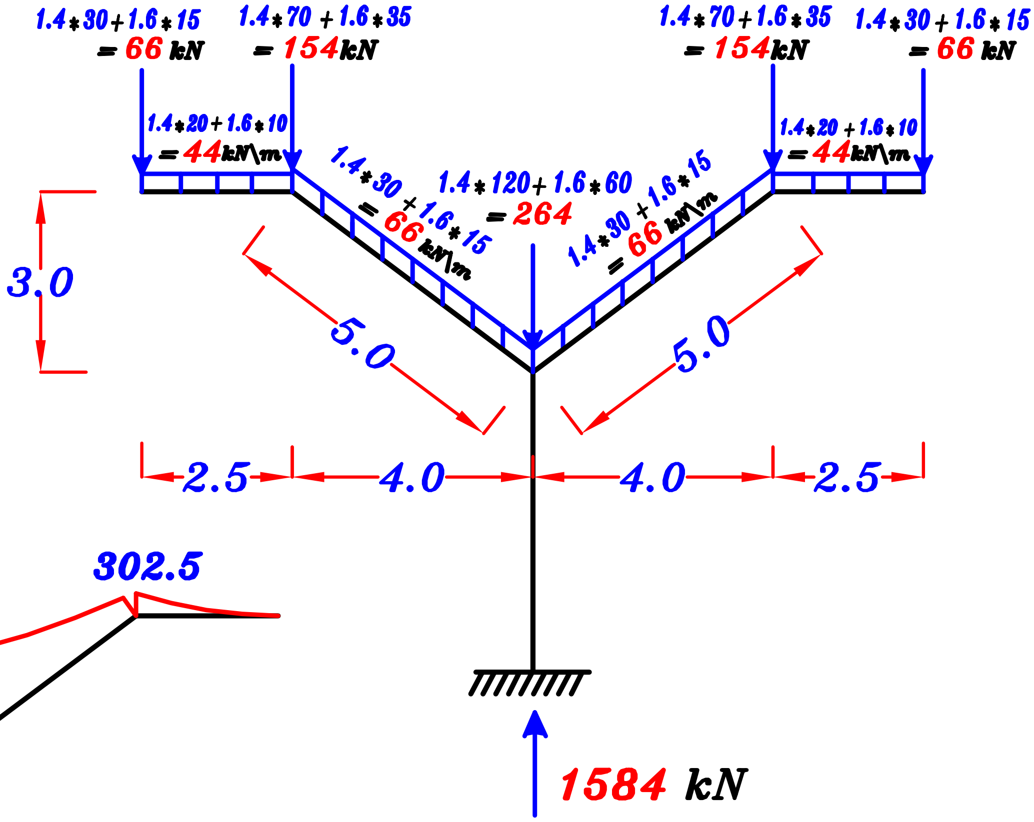
max–max B.M.D.

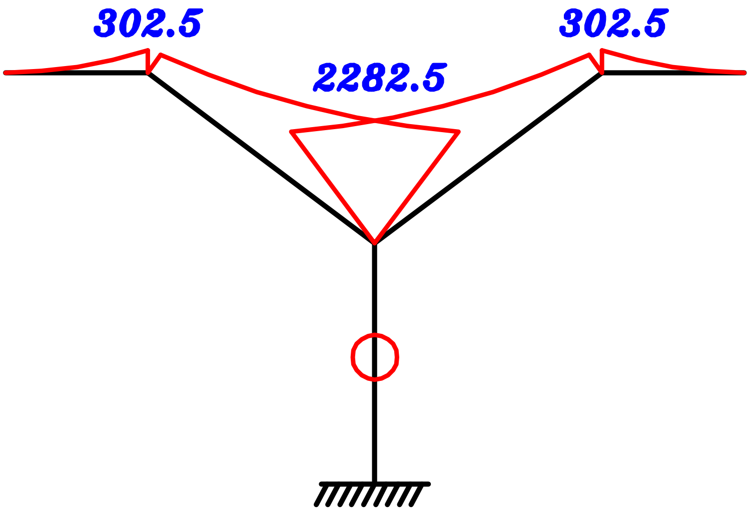

123.7
2282.5
302.5
933.7
1348.7
1348.7
302.5
2282.5
123.7
933.7
1348.7
1348.7


302.5
302.5
2282.5
2
1348.7
1
3
1348.7

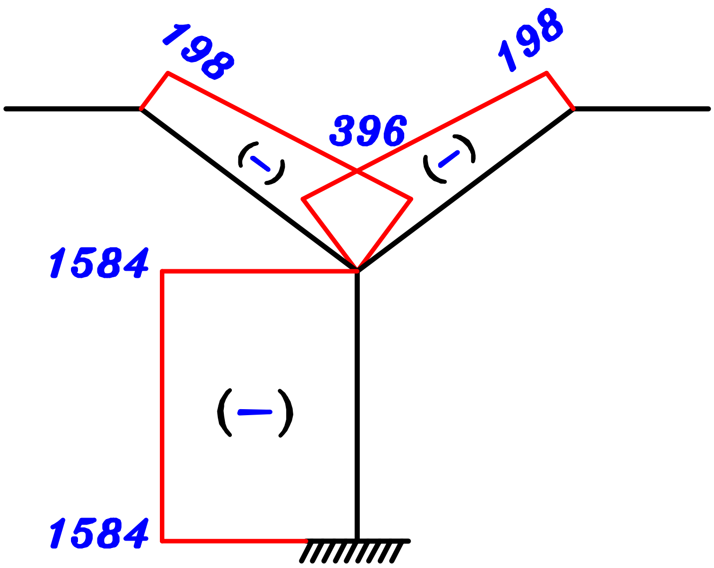
max N.F.D. (Not a critical case)



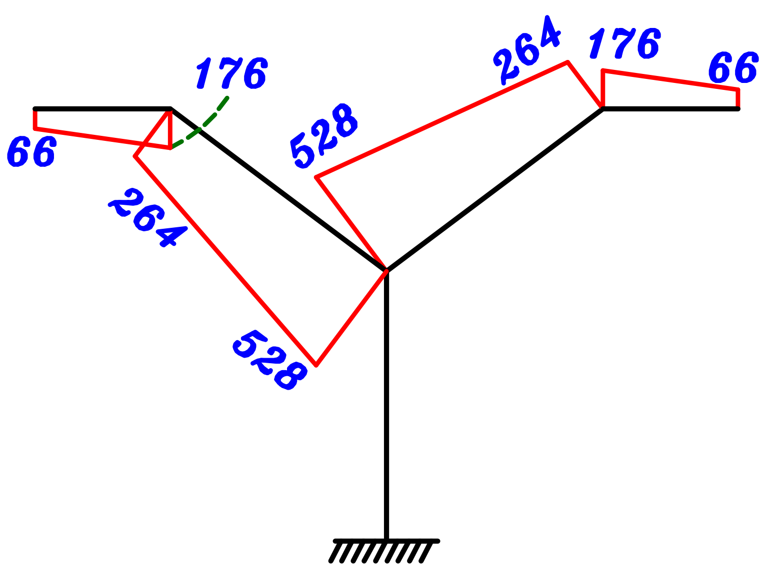




B.M.D.
(U.L.)



N.F.D.
(U.L.)



S.F.D.
(U.L.)

Design of Sections.

Sec. ① $M = 2282.5$ kN.m , $P = 396$ kN , $b = 400$ mm

$$d_o = 3.5\sqrt{\frac{2282.5 * 10^6}{25 * 400}} = 1672.1 \text{ mm} \quad \text{(as R-Sec.)}$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1672.1) = (1839.3 \rightarrow 2173) \text{ mm}$$

Take $d = 1900$ mm , $t = 1900 + 100 = 2000$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{396 * 10^3}{25 * 400 * 2000} = 0.0198 < 0.04 \quad \therefore$ (neglect P)

$\therefore$ Take $d = d_o = 1672.1$ mm

$\therefore$ Take $\boxed{d = 1700 \text{ mm}}$, $\boxed{t = 1800 \text{ mm}}$

$\because$ The sec. still R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{2282.5 * 10^6}{0.780 * 360 * 1672.1} = 4861 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 4861 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1700 = 2125 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 4861 \text{ mm}^2$ $\qquad 10\phi 25$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 4861 \qquad 5\phi 12$

Sec. ② $M_{U.L.} = 302.5$ kN.m R–Sec.

Take $t = \frac{t_1}{2} = \frac{1.80}{2} = 0.90\ m$

$\therefore$ Take $\boxed{d = 850\ mm}$, $\boxed{t = 900\ mm}$

$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\ b}} \quad \therefore 850 = C_1 \sqrt{\frac{302.5 * 10^6}{25 * 400}} \longrightarrow C_1 = 4.88 \longrightarrow J = 0.826$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{302.5 * 10^6}{0.826 * 360 * 850} = 1196\ mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 1196\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 850 = 1062\ mm^2$$

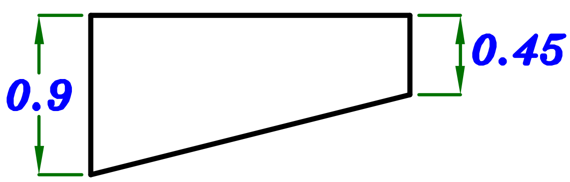
$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1196\ mm^2$ $3 \phi 25$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{400 - 25}{25 + 25} = 7.50 = 7.0 \text{ bars}$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (0.1 \rightarrow 0.2)\ 1196$ $2 \phi 12$

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{900}{2} = 450\ mm \\ t_b = \frac{spacing}{12} = \frac{5000}{12} = 416.6\ mm \\ t - \frac{L_c}{3} = 900 - \frac{2500}{3} = 66.6\ mm \end{array} \right\}$$

$\boxed{Y = 450\ mm}$



Sec. ③ R-Sec. $M = 1348.7$ kN.m , $P = 1194$ kN

$$d_o = 3.5\sqrt{\frac{1348.7 * 10^6}{25 * 400}} = 1285.3 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1285.3) = (1413.8 \rightarrow 1670.9) \text{ mm}$$

$\therefore$ Take $d = 1500$ mm , $t = 1600$ mm

$$\xrightarrow{\text{Take}} \boxed{t_{(Column)} = t_{(Beam)} = 1800 \text{ mm}}$$ للتسهيل

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{1194 \quad * 10^3}{25 * 400 * 1800} = 0.066 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{1348.7}{1194} = 1.129 \text{ m} \quad \therefore \frac{e}{t} = \frac{1.129}{1.80} = 0.627 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 1.129 + \frac{1.80}{2} - 0.10 = 1.929 \text{ m}$$

$$M_s = P * e_s = 1194 * 1.929 = 2303.2 \text{ kN.m}$$

$$\therefore 1700 = C_1 \sqrt{\frac{2303.2 * 10^6}{25 * 400}} \longrightarrow C_1 = 3.54 \longrightarrow J = 0.783$$

$$\because A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{2303.2 * 10^6}{0.783 * 360 * 1700} - \frac{1194 * 10^3}{(360 \backslash 1.15)} = 992.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 992.2 \text{ mm}^2$

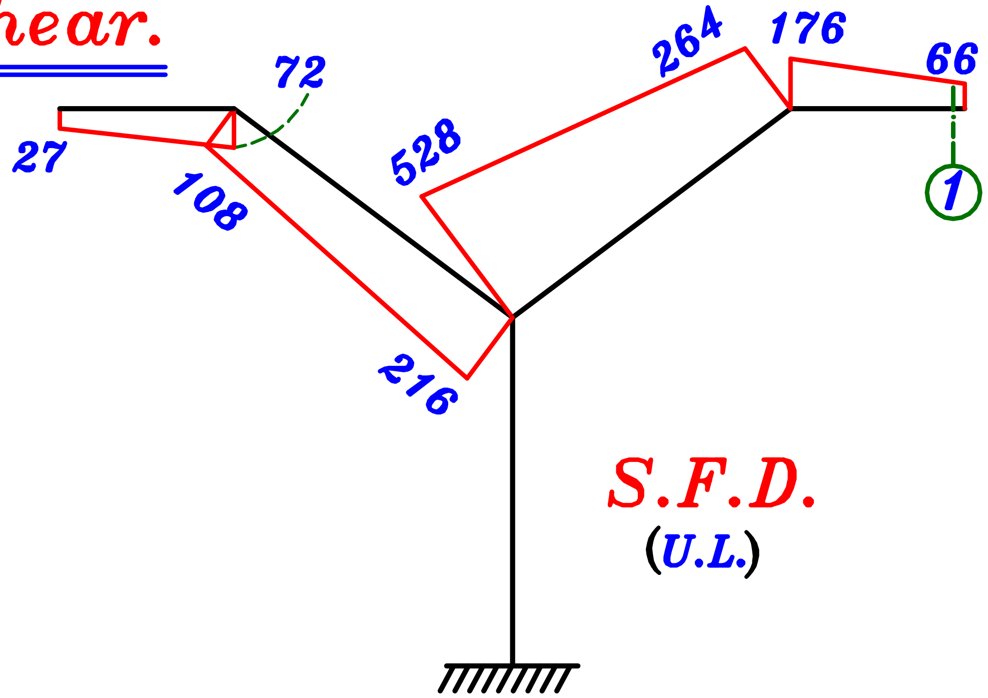
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1700 = 2125 \text{ mm}^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \dfrac{\sqrt{25}}{360}\right) 400 * 1700 = 2125 \text{ mm}^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 992.2 = 1289.9 \text{ mm}^2 \end{array}\right\} = 1289.9 \text{ mm}^2$$ الأقل

$4\, \phi\, 25$

Check shear.

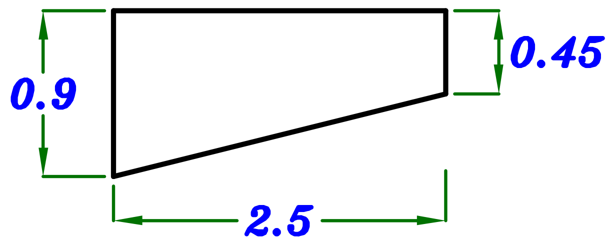


S.F.D.
(U.L.)

- Allowable shear stress.

- $q_{cu} = 0.24 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\dfrac{25}{1.5}} = 0.98 \ N\backslash mm^2$

- $q_{max.} = 0.7 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.7 \sqrt{\dfrac{25}{1.5}} = 2.85 \ N\backslash mm^2$

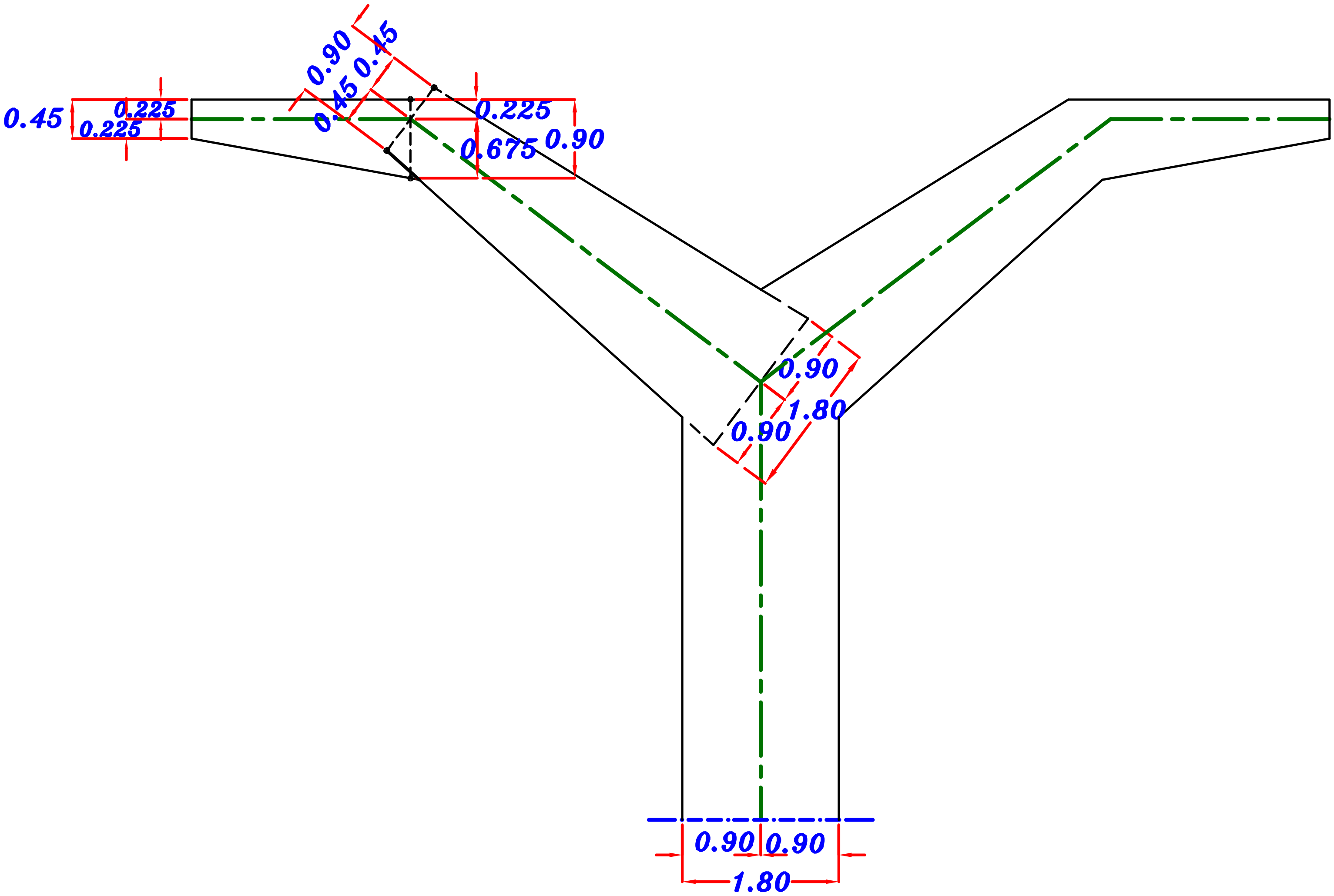


Sec. 1 $Q = 66.0 \ kN \qquad \tan \beta = 0.18$

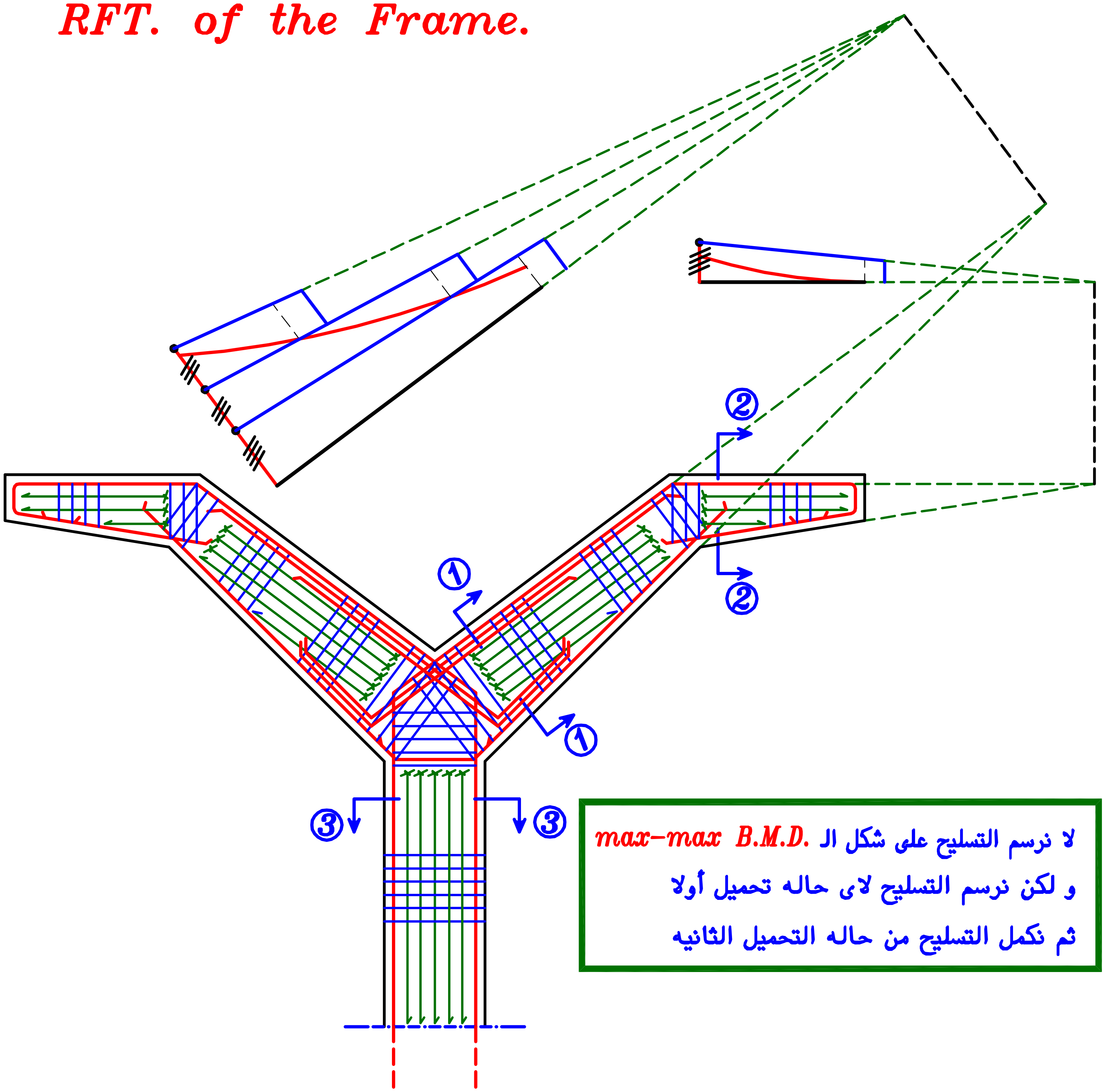
$\therefore$ Actual shear stress. $= q_U = \dfrac{Q}{b\,d} - \dfrac{M \tan \beta}{b \ d^2}$

$$q_U = \frac{66.0 * 10^3}{400 * 400} - ZERO = 0.412 \ N\backslash mm^2$$

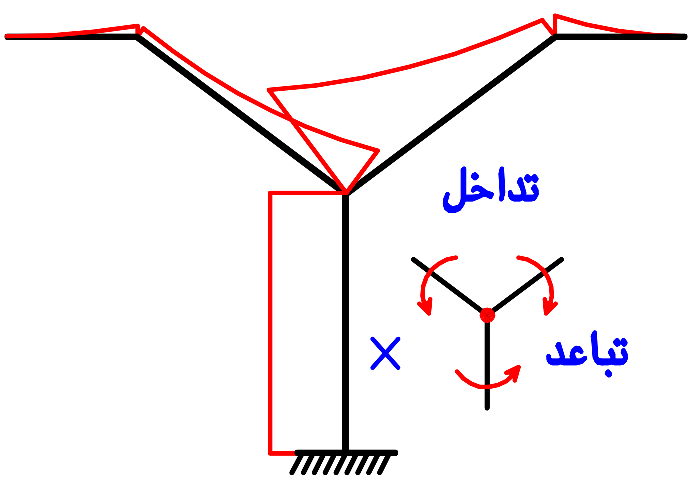
$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m`}$

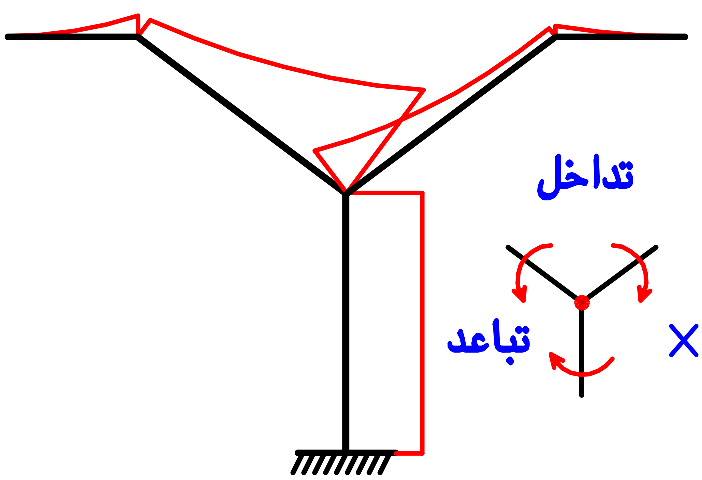

0.45
0.225
0.225
0.90
0.45
0.45
0.225
0.675
0.90
0.90
1.80
0.90
0.90
0.90
1.80

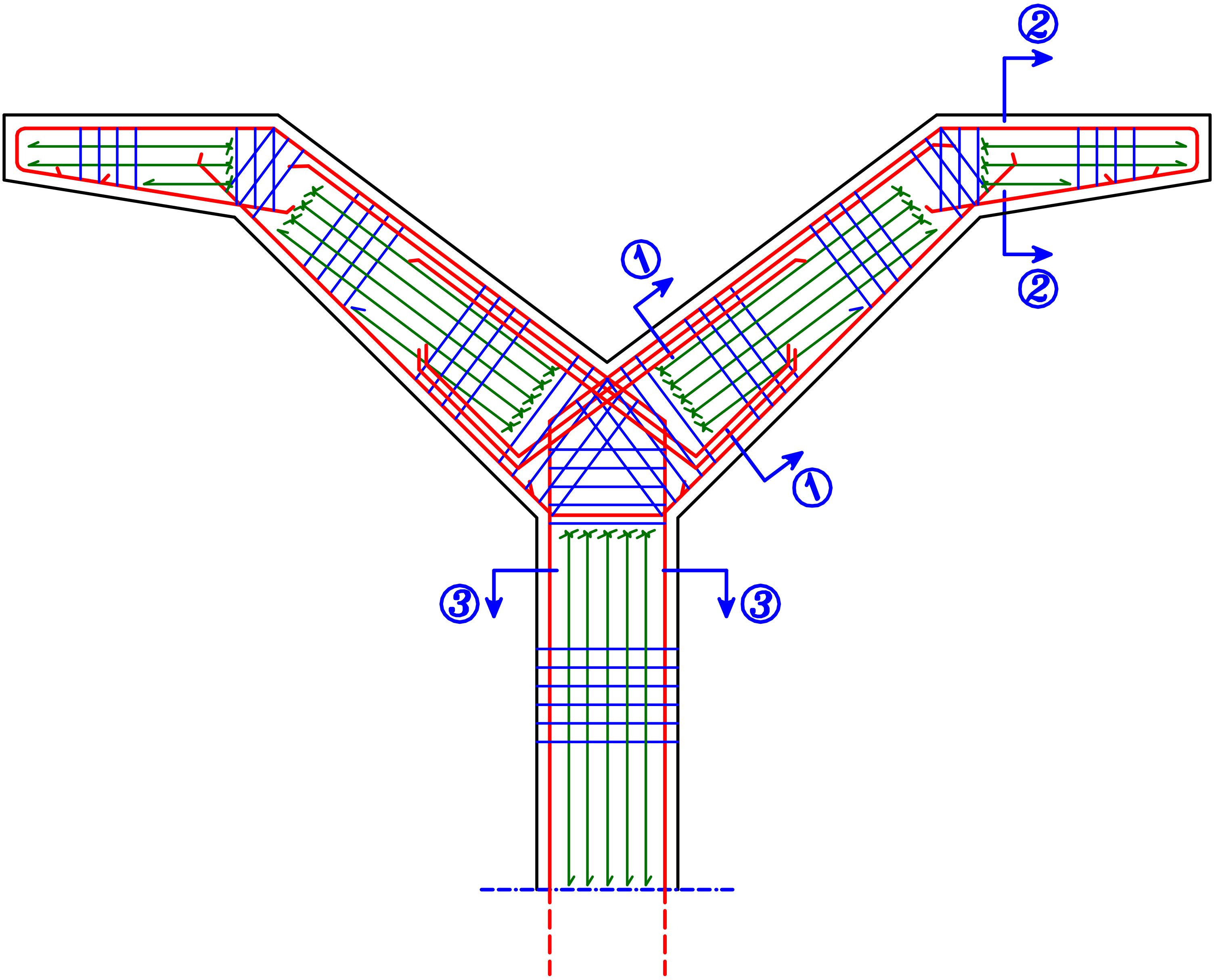
RFT. of the Frame.

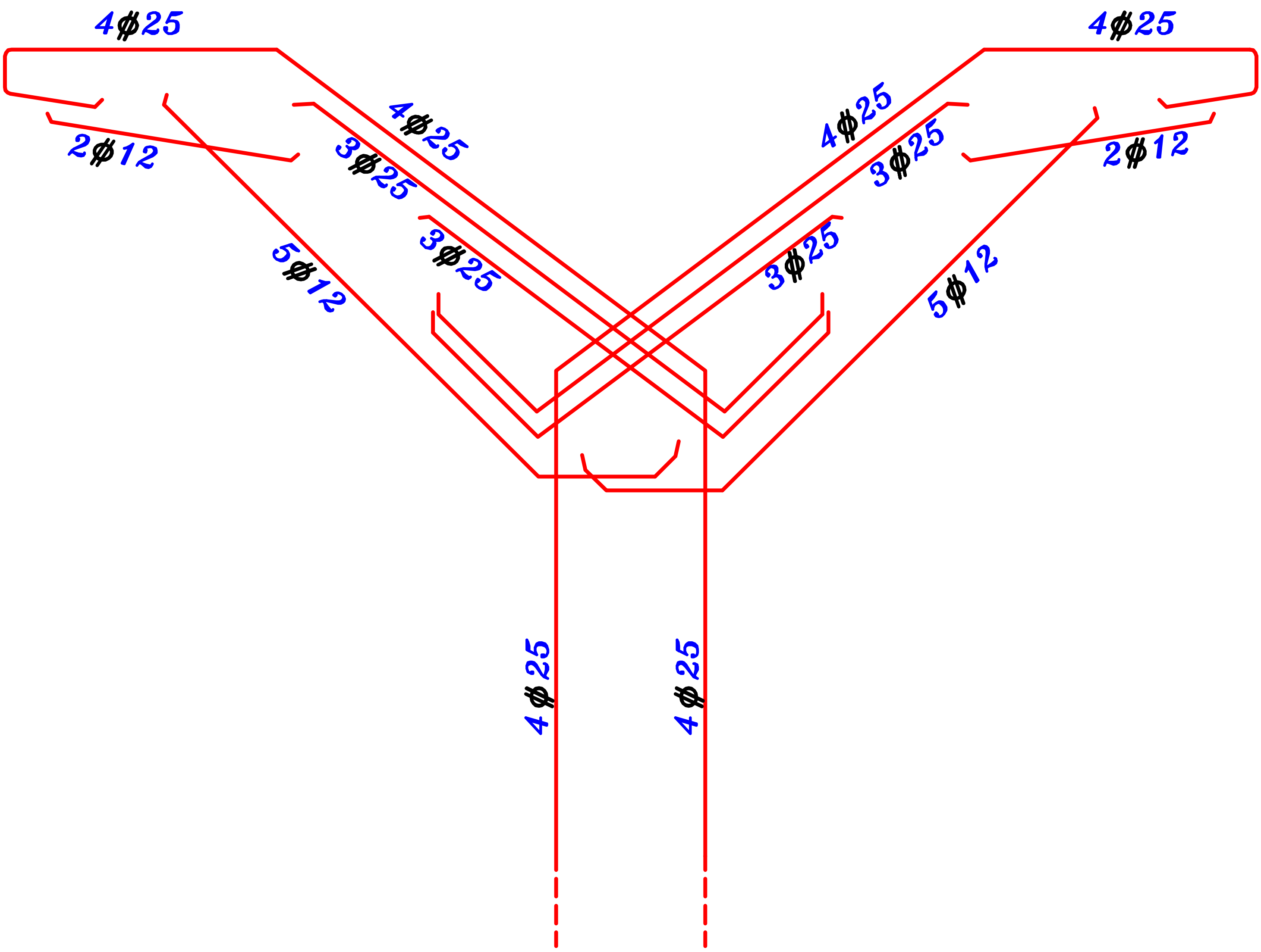


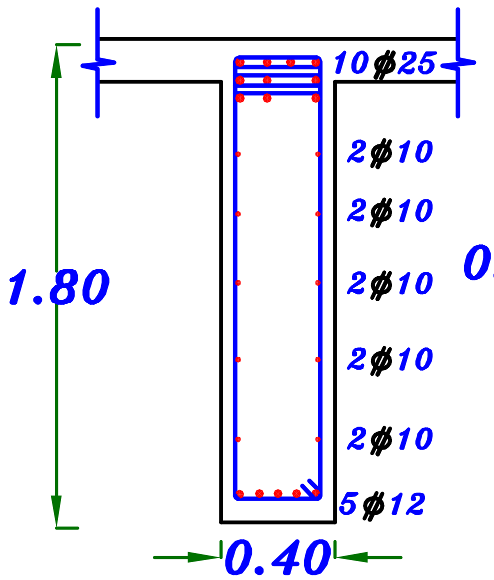
max−max B.M.D. لا نرسم التسليح على شكل الـ
و لكن نرسم التسليح لاى حاله تحميل أولا
ثم نكمل التسليح من حاله التحميل الثانيه



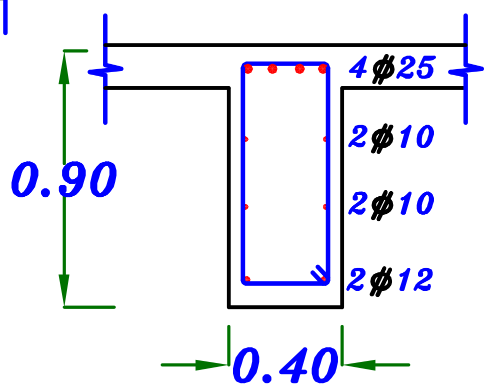



2
2
1
1
3
3

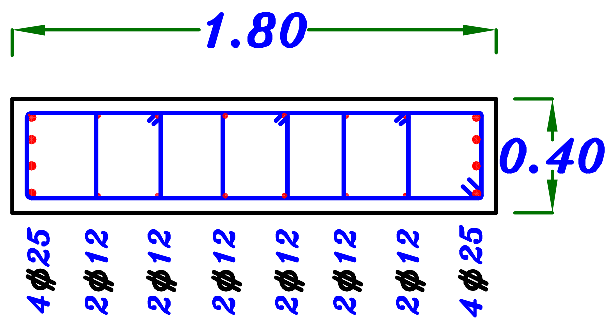




Sec. (1–1)

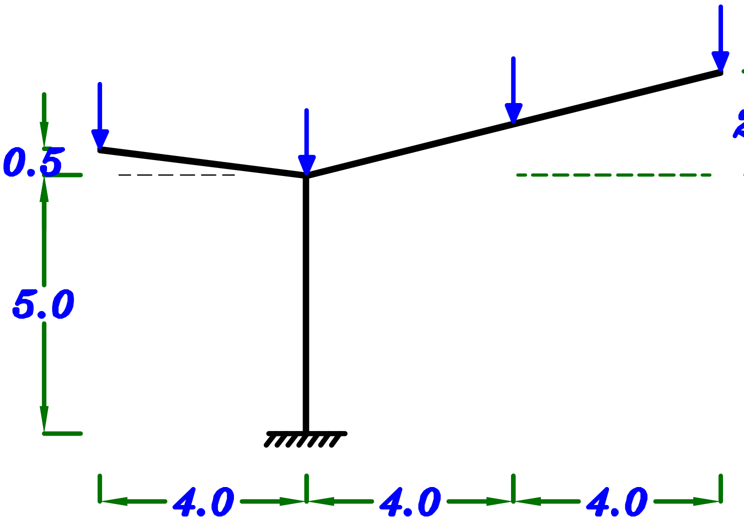


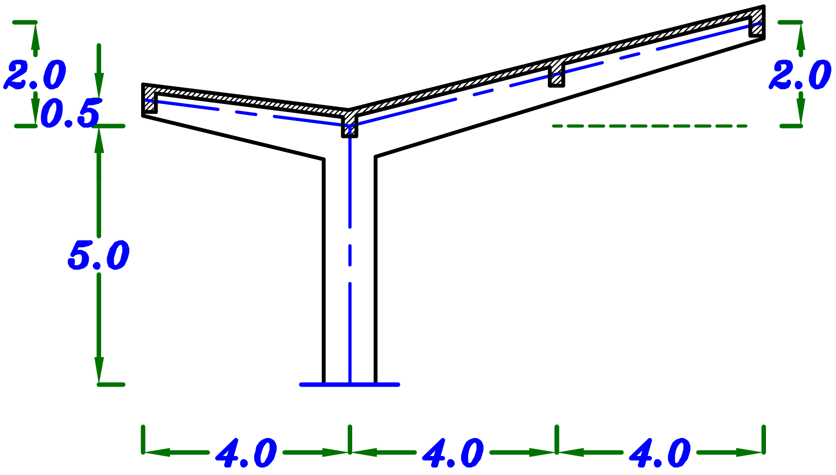
Sec. (2–2)



Sec. (3–3)

Example.





Data.

$F_{cu} = 30 \; N\backslash mm^2 \quad F_y = 400 \; N\backslash mm^2$

$t_s = 140 \; mm \quad L.L. = 1.50 \; kN/m^2 \quad F.C. = 2.0 \; kN/m^2$

$Spacing = 6.0 \; m$

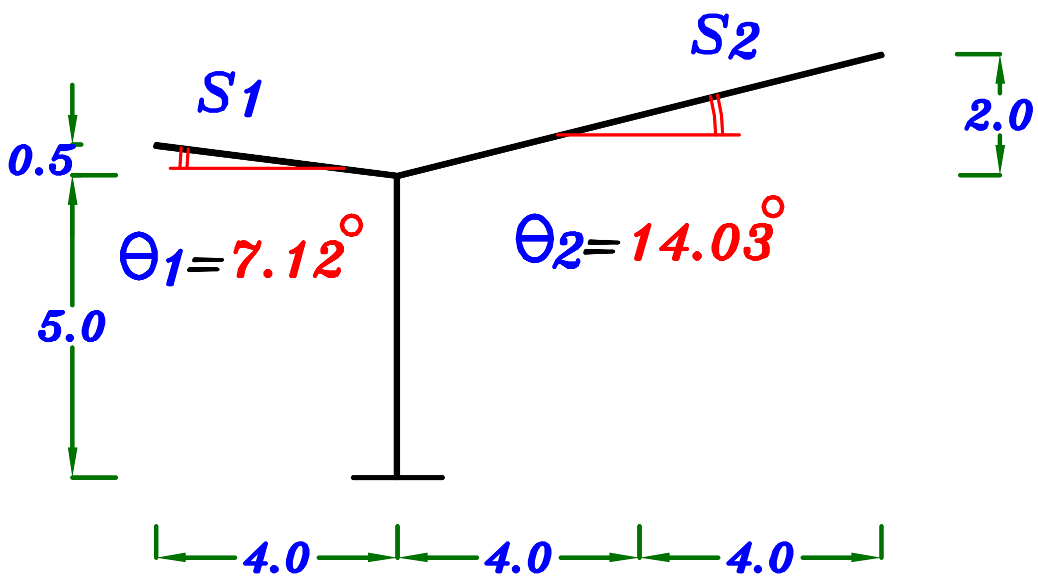
Required.

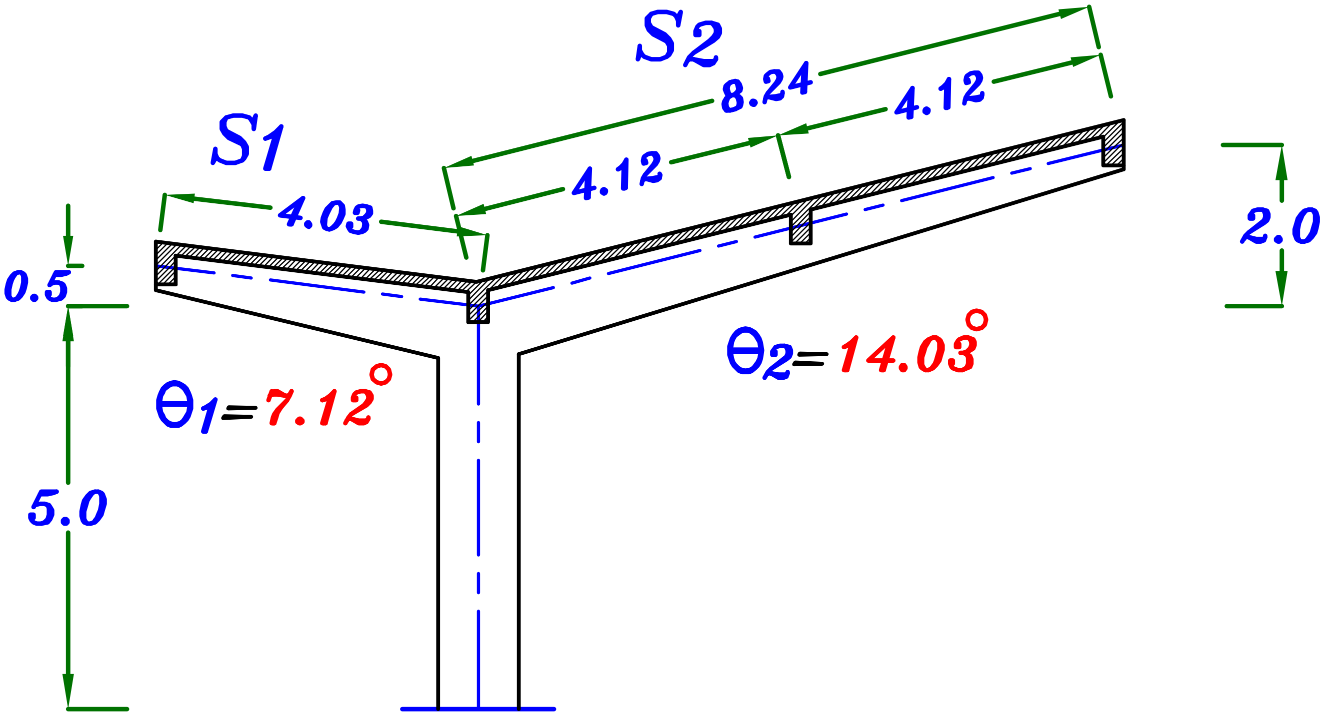
1– Make Load distribution and draw max–max B.M.D. , N.F.D. & S.F.D.

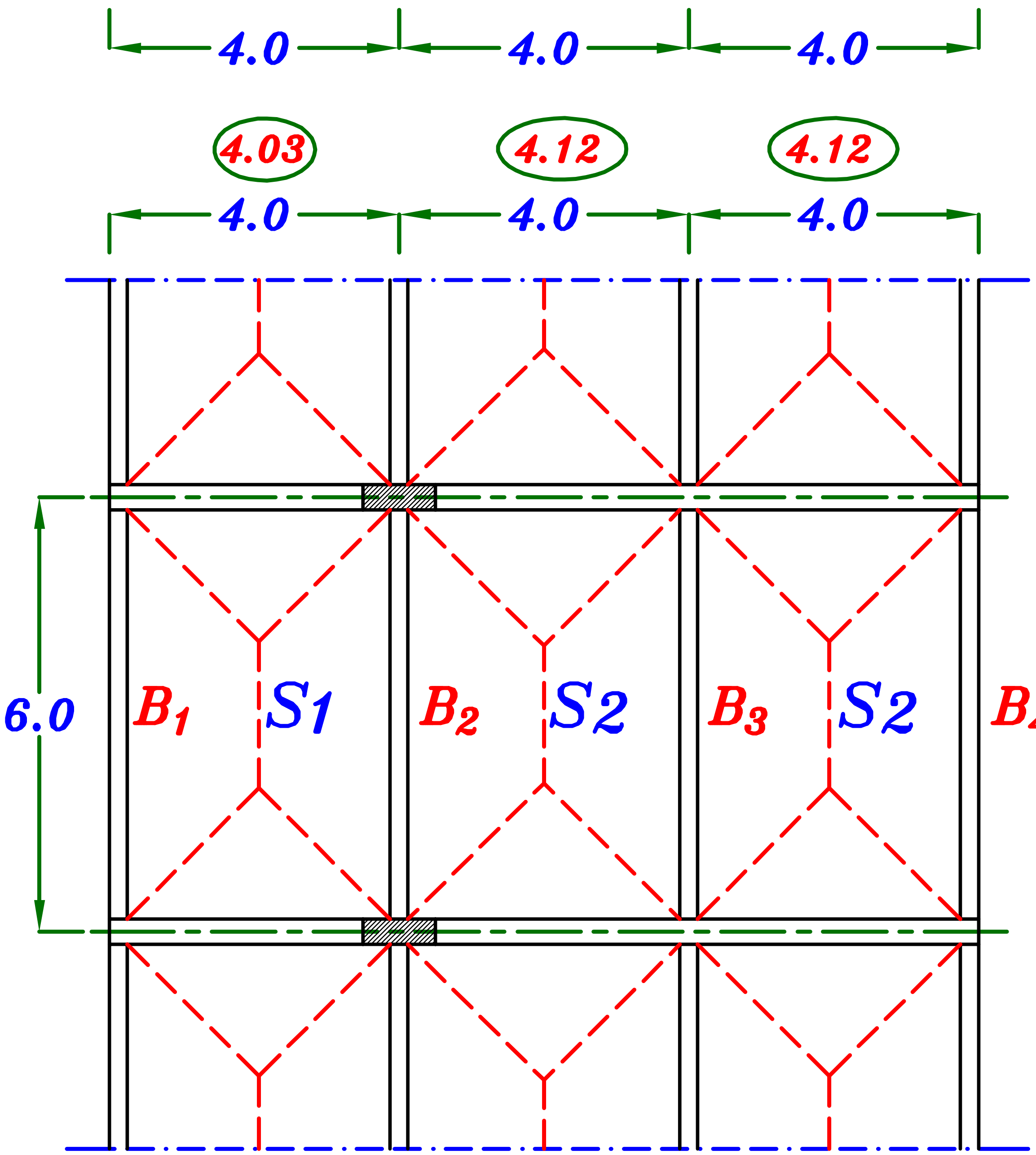
2– Design the critical sections of the Frame.

3– Draw Details of RFT. of the Frame.

Solution.




S2
8.24
4.12
4.12
S1
4.03
0.5
2.0
5.0
θ1=7.12°
θ2=14.03°


4.0
4.0
4.0
4.03
4.12
4.12
4.0
4.0
4.0
6.0
B1
S1
B2
S2
B3
S2
B4

g_s, p_s

$$g_s = t_s * \gamma_c + F.C. = 0.14 * 25 + 2.0 = 5.50 \ kN\backslash m^2$$

$$p_{si1} = L.L. * \cos\theta = 1.5 * \cos 7.12^\circ = 1.49 \ kN\backslash m^2$$ ------ Slab S_1

$$p_{si2} = L.L. * \cos\theta = 1.5 * \cos 14.03^\circ = 1.455 \ kN\backslash m^2$$ ------ Slab S_2

B_1 Load For Shear.

For Trapezoid 1 $C_{a1} = 1 - \frac{1}{2}\left(\frac{L_s}{L}\right) = 1 - \frac{1}{2}\left(\frac{4.03}{6.0}\right) = 0.664$

$$g_a = O.W. + C_{a1}\, g_s \frac{L_s}{2} = 3.0 + (0.664)(5.50)\left(\frac{4.03}{2}\right) = 10.35 \ kN\backslash m'$$

$$p_a = C_{a1}\, p_{si1} \frac{L_s}{2} = (0.664)(1.49)\left(\frac{4.03}{2}\right) = 2.0 \ kN\backslash m'$$

$$w_a = g_a + p_a = 10.35 + 2.0 = 12.35 \ kN\backslash m'$$

$$R_1 = g_a * Spacing = 10.35 * 6.0 = 62.1 \ kN$$ ------ D.L.

$$= w_a * Spacing = 12.35 * 6.0 = 74.1 \ kN$$ ------ T.L.

$$\boxed{\begin{aligned} R_1 &= 62.1 \ kN \text{ ---- D.L.} \\ &= 74.1 \ kN \text{ ---- T.L.} \end{aligned}}$$

B_2 Load For Shear.

$C_{a1} = 0.664$

For Trapezoid 2 $C_{a2} = 1 - \frac{1}{2}\left(\frac{L_s}{L}\right) = 1 - \frac{1}{2}\left(\frac{4.12}{6.0}\right) = 0.656$

$$g_a = O.W. + C_{a1}\, g_s \frac{L_s}{2} + C_{a2}\, g_s \frac{L_s}{2}$$

$$= 3.0 + (0.664)(5.50)\left(\frac{4.03}{2}\right) + (0.656)(5.50)\left(\frac{4.12}{2}\right) = 17.79 \ kN\backslash m'$$

$$p_a = C_{a1}\, p_{si1} \frac{L_s}{2} + C_{a2}\, p_{si2} \frac{L_s}{2} = (0.664)(1.49)\left(\frac{4.03}{2}\right) + (0.656)(1.455)\left(\frac{4.12}{2}\right) = 3.96 \ kN\backslash m'$$

$$w_a = g_a + p_a = 17.79 + 3.96 = 21.75 \ kN\backslash m'$$

$$R_2 = g_a * Spacing = 17.79 * 6.0 = 106.74 \ kN$$ ------ D.L.

$$= w_a * Spacing = 21.75 * 6.0 = 130.5 \ kN$$ ------ T.L.

$$\boxed{\begin{aligned} R_2 &= 106.74 \ kN \text{ --- D.L.} \\ &= 130.5 \ kN \text{ --- T.L.} \end{aligned}}$$

B_3 Load For Shear.

For Trapezoid 2 $C_{a2} = 0.656$

$$g_a = O.W. + 2\,C_{a2}\,g_s\,\frac{L_s}{2} = 3.0 + 2\,(0.656)\,(5.50)\,\left(\frac{4.12}{2}\right) = 17.86 \ kN\backslash m'$$

$$p_a = 2\,C_{a2}\,p_{si2}\,\frac{L_s}{2} = 2\,(0.656)\,(1.455)\,\left(\frac{4.12}{2}\right) = 3.93 \ kN\backslash m'$$

$$w_a = g_a + p_a = 17.86 + 3.93 = 21.79 \ kN\backslash m'$$

$$R_3 = g_a * Spacing = 17.86 * 6.0 = 107.16 \ kN \ \text{------ D.L.}$$

$$= w_a * Spacing = 21.79 * 6.0 = 130.74 \ kN \ \text{------ T.L.}$$

$$\boxed{\begin{aligned} R_3 &= 107.16 \ kN \ \text{--- D.L.} \\ &= 130.74 \ kN \ \text{--- T.L.} \end{aligned}}$$

B_4 Load For Shear.

For Trapezoid 2 $C_{a2} = 0.656$

$$g_a = O.W. + C_{a2}\,g_s\,\frac{L_s}{2} = 3.0 + (0.656)\,(5.50)\,\left(\frac{4.12}{2}\right) = 10.43 \ kN\backslash m'$$

$$p_a = C_{a2}\,p_{si2}\,\frac{L_s}{2} = (0.656)\,(1.455)\,\left(\frac{4.12}{2}\right) = 1.96 \ kN\backslash m'$$

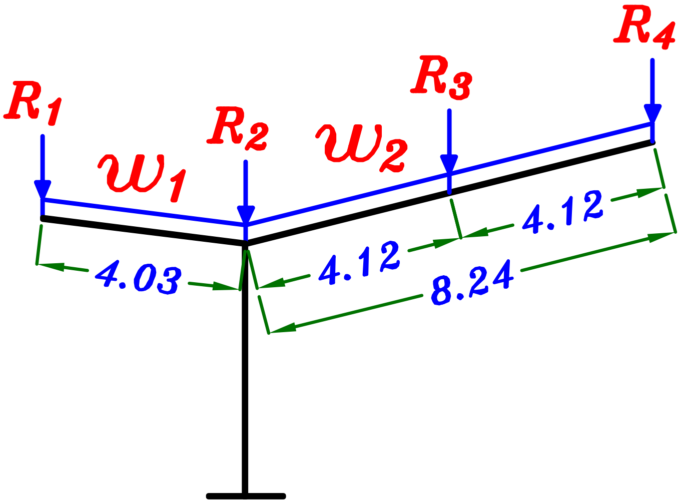
$$w_a = g_a + p_a = 10.43 + 1.96 = 12.39 \ kN\backslash m'$$

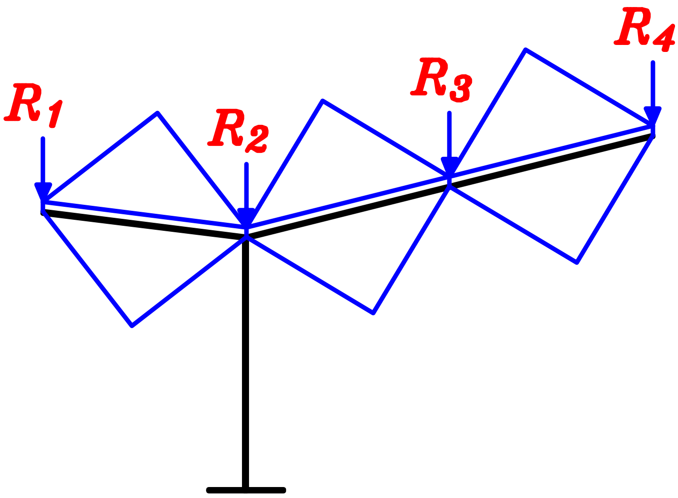
$$R_4 = g_a * Spacing = 10.43 * 6.0 = 62.58 \ kN \ \text{------ D.L.}$$

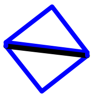
$$= w_a * Spacing = 12.39 * 6.0 = 74.34 \ kN \ \text{------ T.L.}$$

$$\boxed{\begin{aligned} R_4 &= 62.58 \ kN \ \text{--- D.L.} \\ &= 74.34 \ kN \ \text{--- T.L.} \end{aligned}}$$

Loads on the Frame.







(W1)

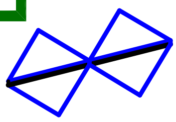
$$\frac{\sum area}{span} = \frac{2\left(\frac{1}{2}(4.03)\left(\frac{4.03}{2}\right)\right)}{4.03} = 2.015$$

$$g_1 = g_a = g_e = o.w. + \frac{\sum area}{span} * g_s = 6.0 + 2.015\,(5.50) = 17.08 \; kN\backslash m$$

$$p_1 = p_a = p_e = \frac{\sum area}{span} * p_{si1} = 2.015\,(1.49) = 3.0 \; kN\backslash m'$$

$$w_1 = w_a = w_e = g_1 + p_1 = 17.08 + 3.0 = 20.08 \; kN\backslash m'$$

$$\boxed{\begin{array}{l} g_1 = 17.08 \; kN\backslash m' \text{ --- D.L.} \\ w_1 = 20.08 \; kN\backslash m' \text{ --- T.L.} \end{array}}$$



(W2)

$$\frac{\sum area}{span} = \frac{4\left(\frac{1}{2}(4.12)\left(\frac{4.12}{2}\right)\right)}{8.24} = 2.06$$

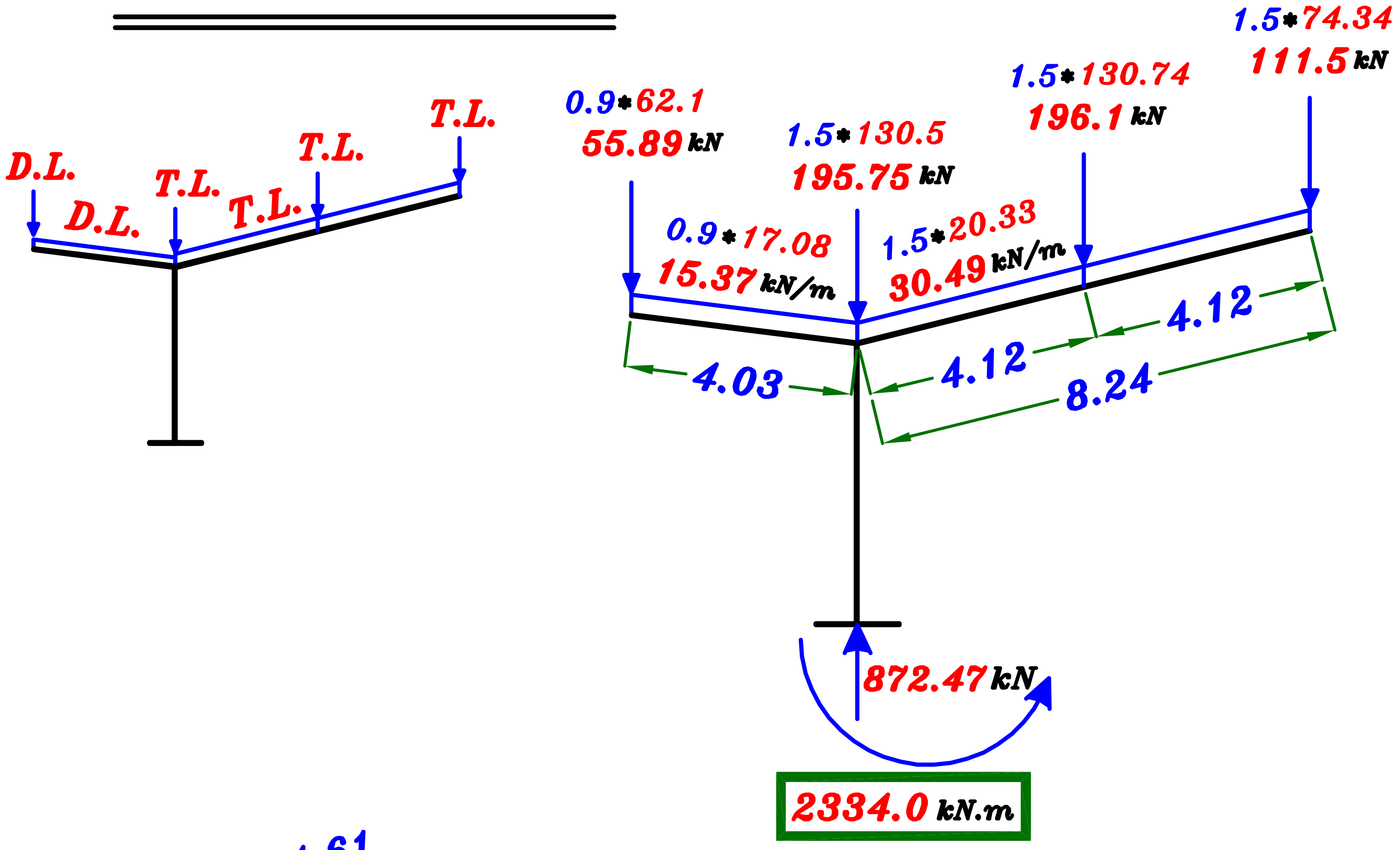
$$g_2 = g_a = g_e = o.w. + \frac{\sum area}{span} * g_s = 6.0 + 2.06\,(5.50) = 17.33 \; kN\backslash m$$

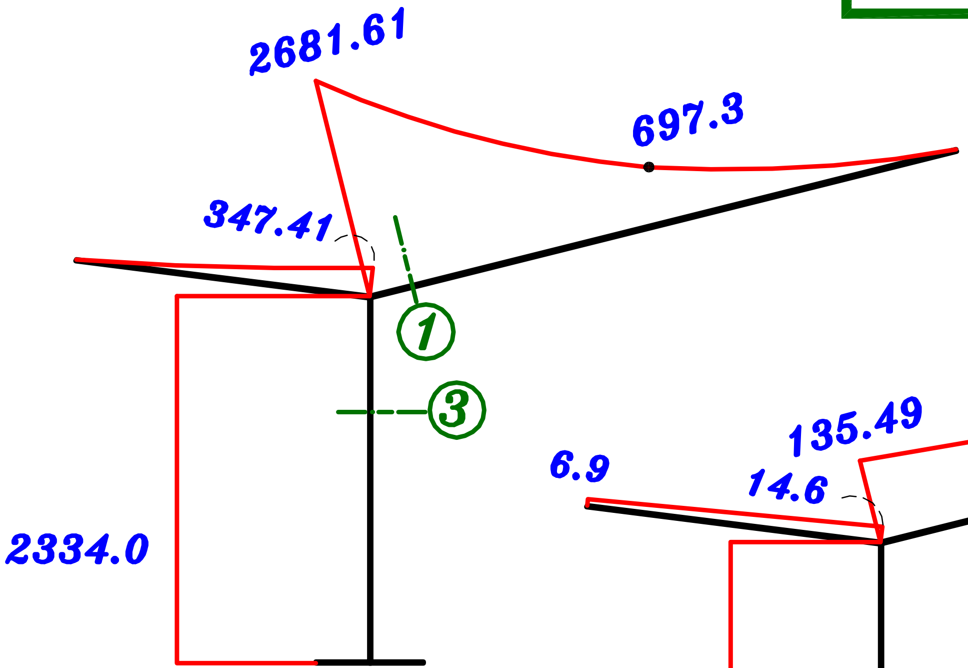
$$p_2 = p_a = p_e = \frac{\sum area}{span} * p_{si2} = 2.06\,(1.455) = 3.0 \; kN\backslash m'$$

$$w_2 = w_a = w_e = g_1 + p_1 = 17.33 + 3.0 = 20.33 \; kN\backslash m'$$

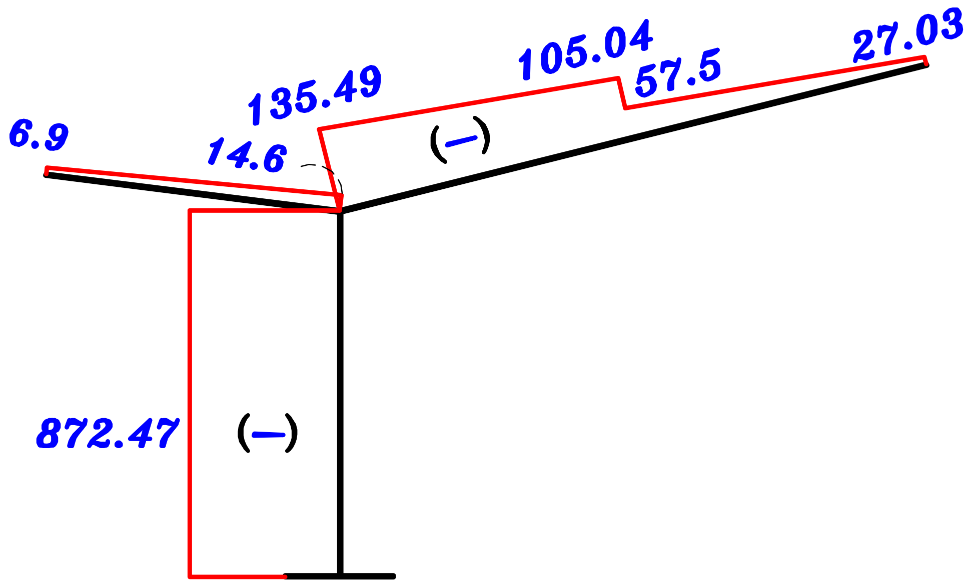
$$\boxed{\begin{array}{l} g_2 = 17.33 \; kN\backslash m' \text{ --- D.L.} \\ w_2 = 20.33 \; kN\backslash m' \text{ --- T.L.} \end{array}}$$

1– max (–Ve) B.M.D.



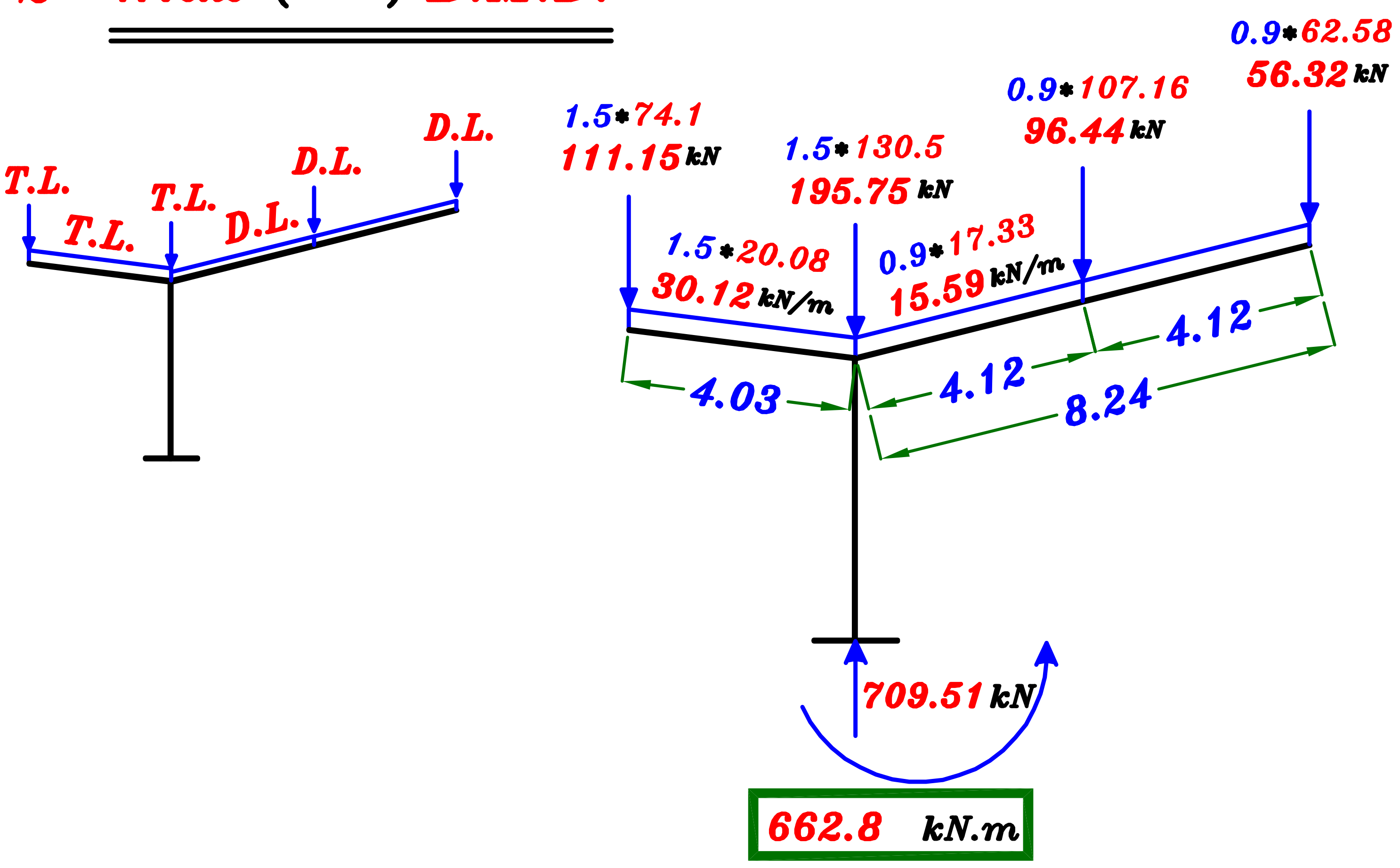


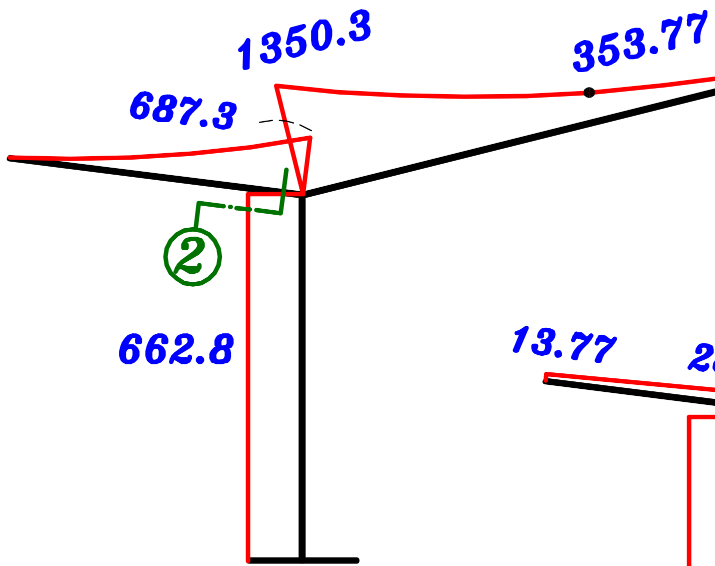
B.M.D.
(U.L.)



N.F.D.
(U.L.)

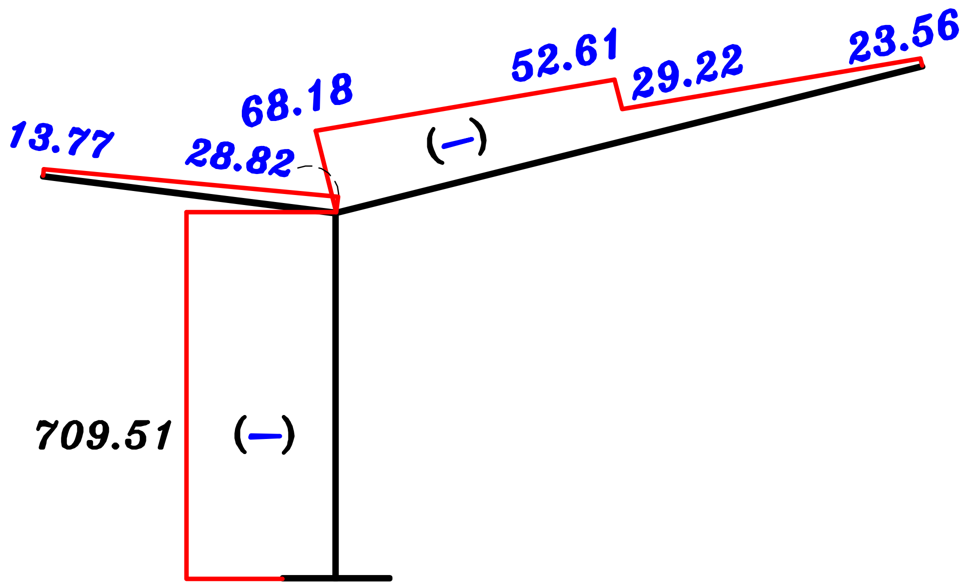
2- max (-Ve) B.M.D.





B.M.D.

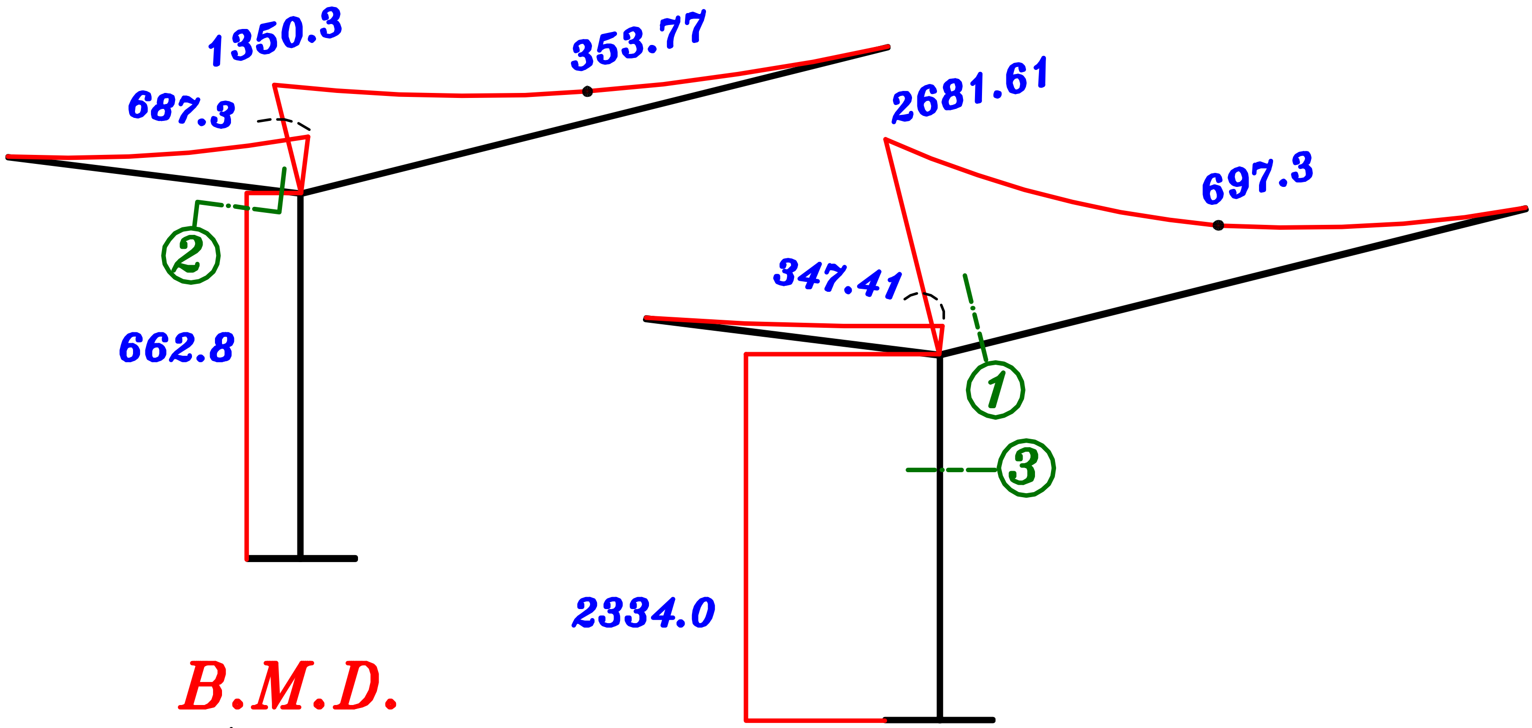
(U.L.)



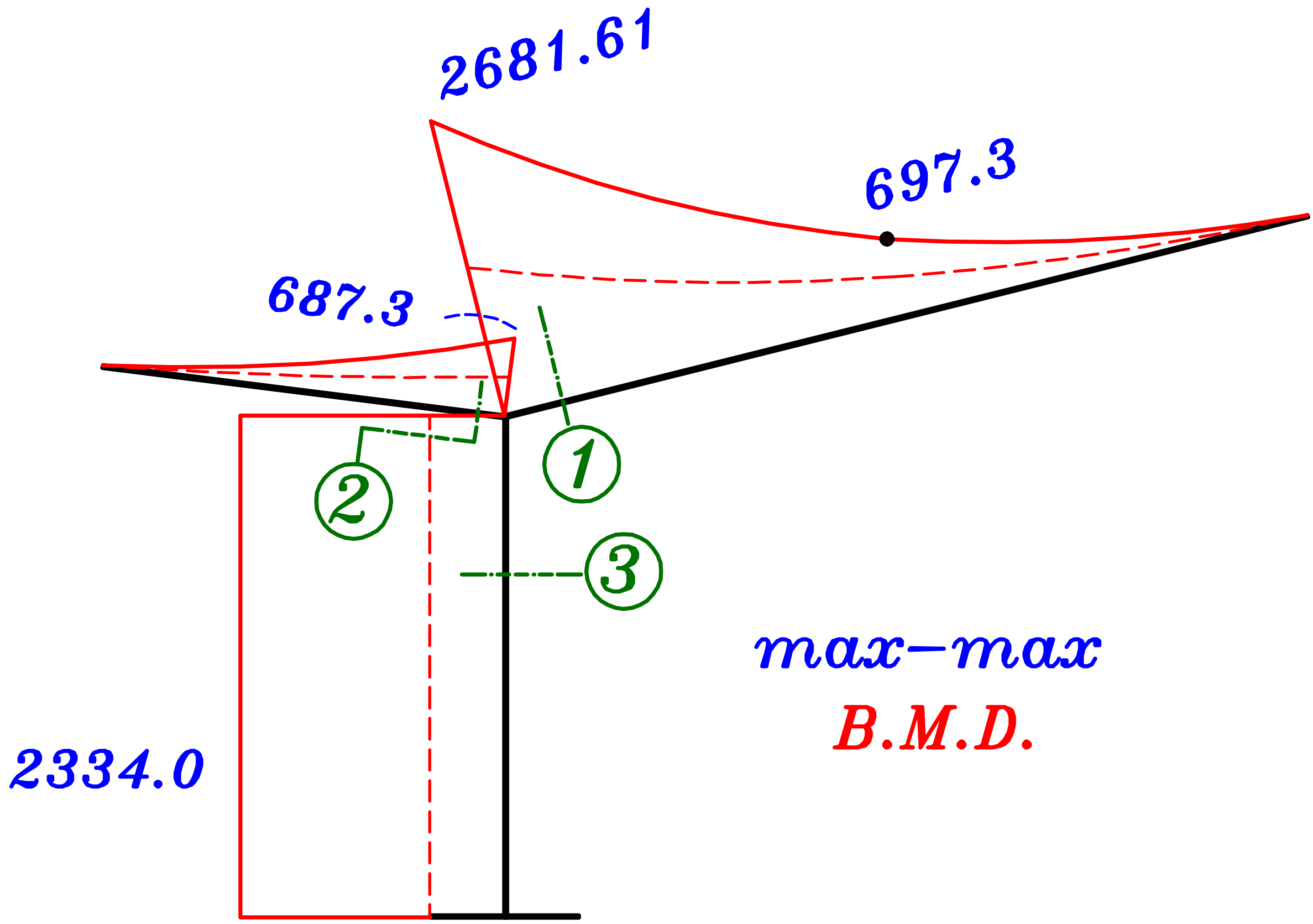
N.F.D.

(U.L.)

max−max B.M.D.

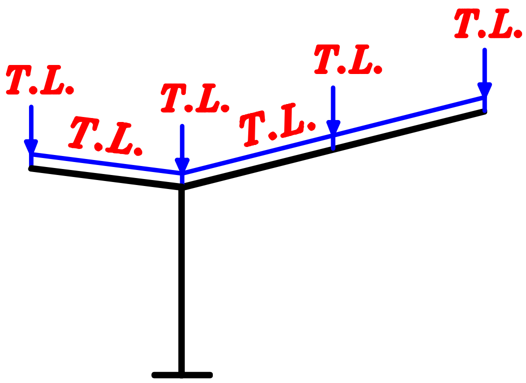


B.M.D.
(U.L.)

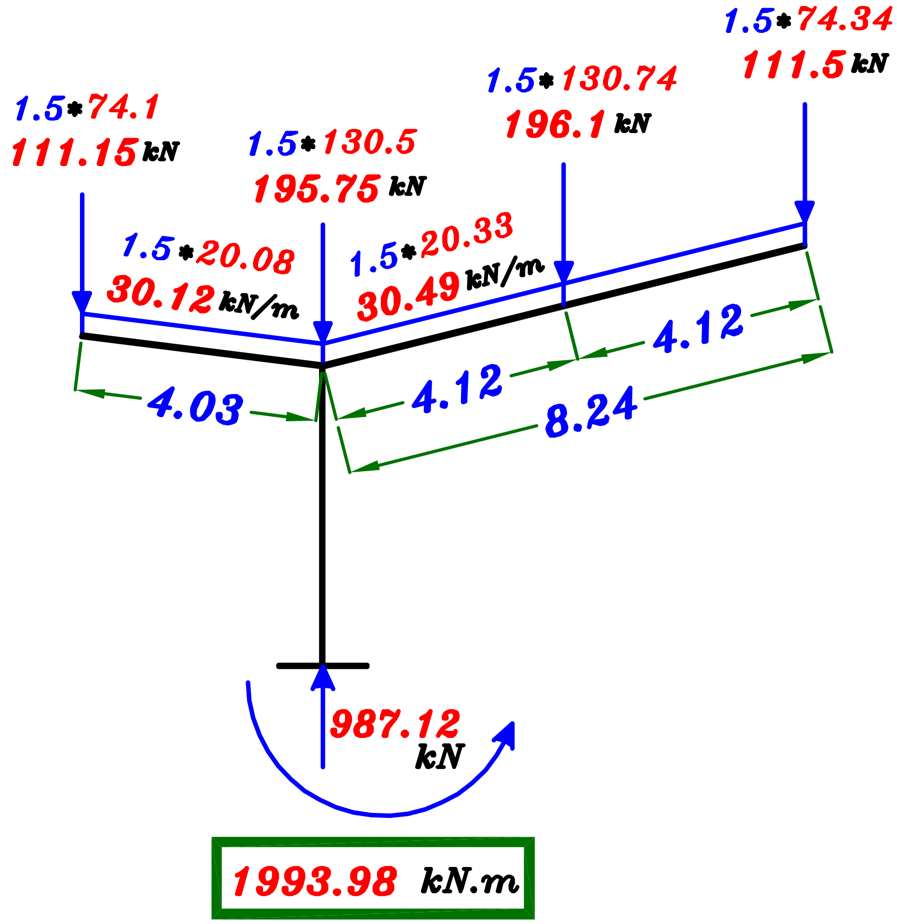


max−max
B.M.D.

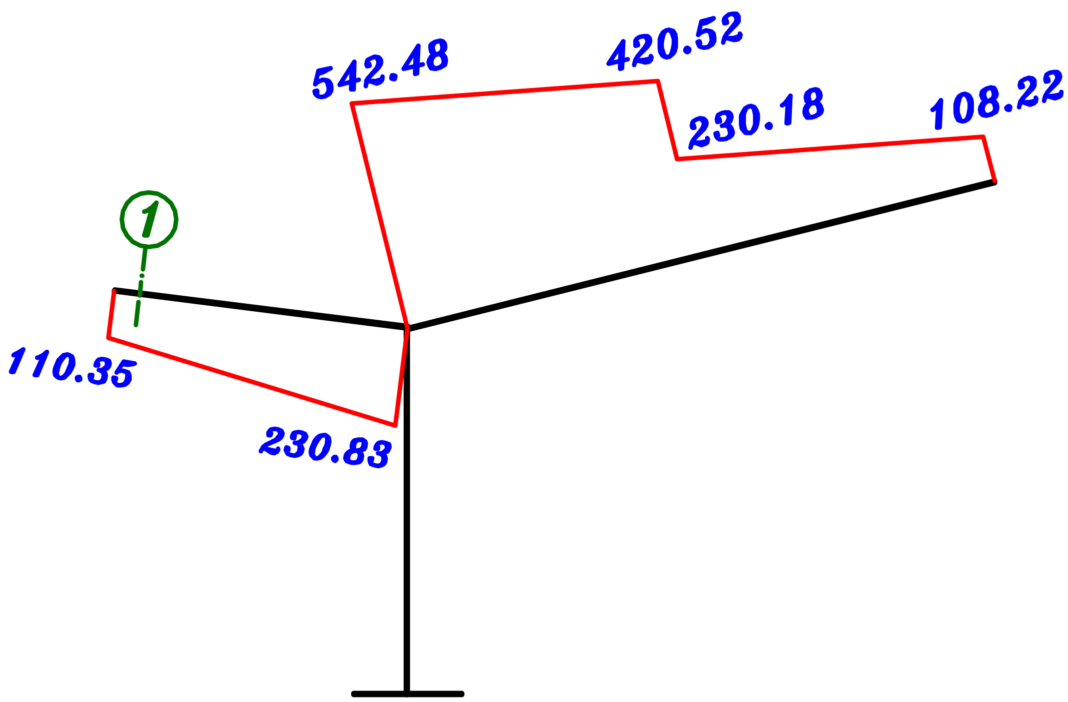
S.F.D.



T.L.
T.L.
T.L.
T.L.
T.L.
T.L.



1.5*74.1
111.15 kN
1.5*130.5
195.75 kN
1.5*130.74
196.1 kN
1.5*74.34
111.5 kN
1.5*20.08
30.12 kN/m
1.5*20.33
30.49 kN/m
4.03
4.12
4.12
8.24
987.12
kN
1993.98 kN.m



542.48
420.52
230.18
108.22
1
110.35
230.83

S.F.D.
(U.L.)

Design of Sections.

Sec. 1 $M = 2681.61\ kN.m$, $P = 135.49\ kN$, $b = 350\ mm$

$$d_o = 3.5\sqrt{\frac{2681.61 * 10^6}{30 * 350}} = 1768.7\ mm \qquad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\ d_o = (1.1 \rightarrow 1.3)\ (1768.7) = (1945.5 \rightarrow 2299)\ mm$$

Take $d = 2000\ mm$, $t = 2000 + 100 = 2100\ mm$

Check $$\frac{P}{F_{cu}\ b\ t} = \frac{135.49 * 10^3}{30 * 350 * 2100} = 0.0062 < 0.04 \quad \therefore (\text{neglect } P)$$

$\therefore$ Take $d = d_o = 1768.7\ mm$

$\therefore$ Take $\boxed{d = 1800\ mm}$, $\boxed{t = 1900\ mm}$

$\because$ The sec. still R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{2681.61 * 10^6}{0.780 * 400 * 1768.7} = 4859.4\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 4859.4\ mm^2$

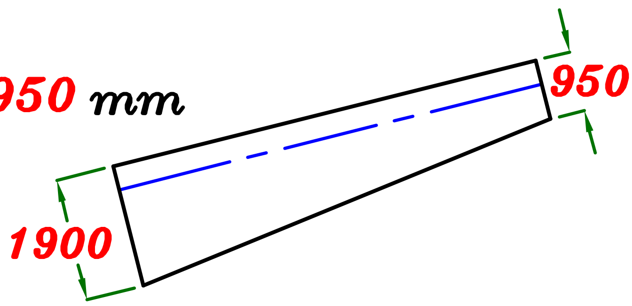
$$\mu_{min.}\ b\ d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\ d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 350 * 1800 = 1941\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\ b\ d \quad \therefore$ Take $A_s = A_{s_{req.}} = 4859.4\ mm^2$ **10 ϕ 25**

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{25 + 25} = 6.50 = 6.0 \text{ bars}$$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (0.1 \rightarrow 0.2)\ 4859.4$ **5 ϕ 12**

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{1900}{2} = 950\ mm \\ t_b = \frac{\text{spacing}}{12} = \frac{6000}{12} = 500\ mm \end{array} \right\} = 950\ mm$$



Sec. ② $M = 687.3$ kN.m , $P = 28.82$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{687.3 * 10^6}{30 * 350}} = 895.0 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \to 1.3)\, d_o = (1.1 \to 1.3)\,(895.4) = (985.0 \to 1164.1) \text{ mm}$$

Take $d = 1000$ mm , $t = 1000 + 100 = 1100$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{28.82 * 10^3}{30 * 350 * 1100} = 0.0025 < 0.04 \quad \therefore (\text{neglect } P)$

$\therefore$ Take $d = d_o = 895.0$ mm

$\therefore$ Take $\boxed{d = 900 \text{ mm}}$, $\boxed{t = 950 \text{ mm}}$

$\because$ The sec. still R-sec. $C_1 = 3.50 \longrightarrow J = 0.78$

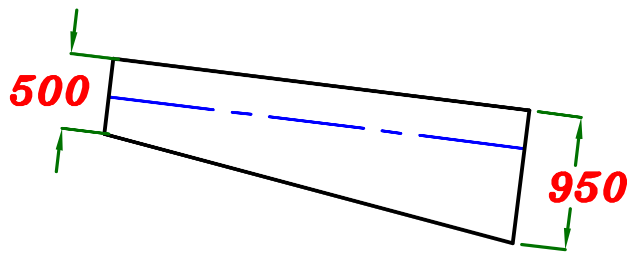
$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{687.3 * 10^6}{0.780 * 400 * 895.0} = 2461.32 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 2461.32 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 350 * 900 = 970.5 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2461.32 \text{ mm}^2$ $\quad$ 6 ϕ25

$$Y = \left\{ \begin{array}{l} \frac{t}{2} = \frac{950}{2} = 475 \text{ mm} \\ t_b = \frac{\text{spacing}}{12} = \frac{6000}{12} = 500 \text{ mm} \end{array} \right\} = 500 \text{ mm}$$



Sec. ③ $M = 2334.0$ kN.m , $P = 872.47$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{2334.0 * 10^6}{30 * 350}} = 1650.2 \text{ mm} \quad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1650.2) = (1815.2 \rightarrow 2145.2) \text{mm}$$

Take $d = 1900$ mm , $t = 1900 + 100 = 2000$ mm

Check $\frac{P}{F_{cu}\, b\, t} = \frac{872.47 * 10^3}{30 * 350 * 2000} = 0.0415 > 0.04$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$$e = \frac{M}{P} = \frac{2334.0}{872.47} = 2.67 \text{ m} \quad \therefore \frac{e}{t} = \frac{2.67}{2.0} = 1.33 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e + \frac{t}{2} - c = 2.67 + \frac{2.0}{2} - 0.10 = 3.57 \text{ m}$$

$$M_s = P * e_s = 872.47 * 3.57 = 3114.7 \text{ kN.m}$$

$$\therefore 1900 = C_1\sqrt{\frac{3114.7 * 10^6}{30 * 350}} \longrightarrow C_1 = 3.48 \longrightarrow J = 0.779$$

$$A_s = \frac{M_s}{J\, F_y\, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{3114.7 * 10^6}{0.779 * 400 * 1900} - \frac{872.47 * 10^3}{(400 \backslash 1.15)} = 2752.6 \text{ mm}^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 2752.6$ mm^2

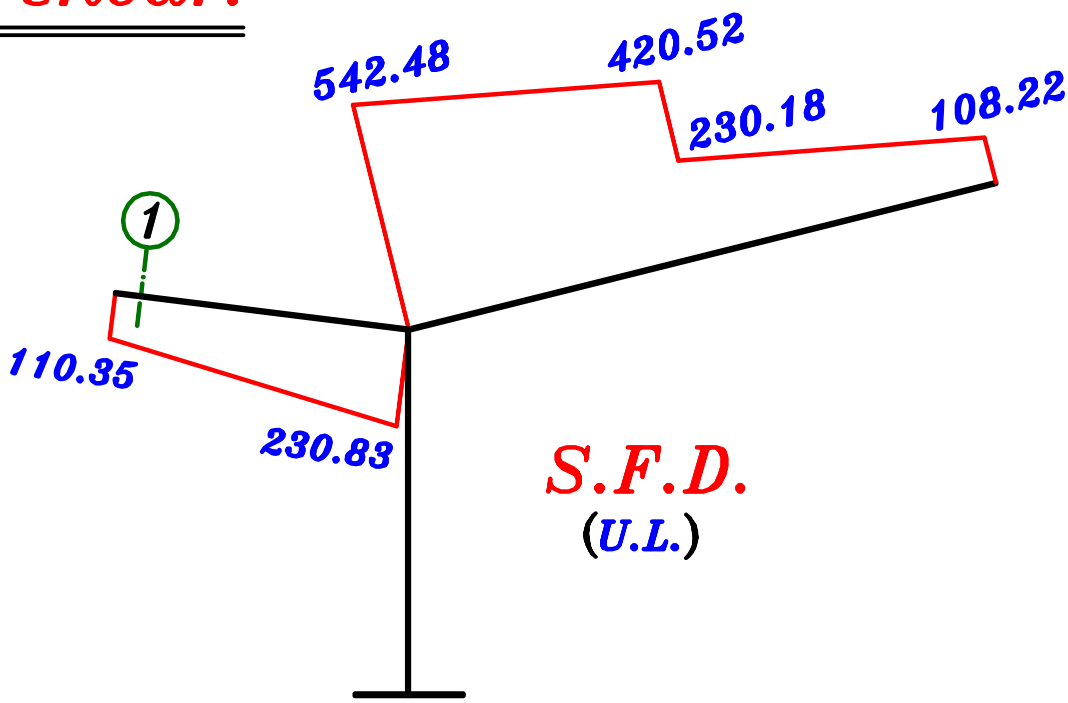
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{400}\right) 350 * 1900 = 2048.8 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 2752.6$ mm^2 6 ϕ25

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{25 + 25} = 6.50 = 6.0 \text{ bars}$$

Stirrup Hangers $\simeq 0.4 A_s \simeq 0.4\,(2752.6) = 1101$ mm^2 3 ϕ25

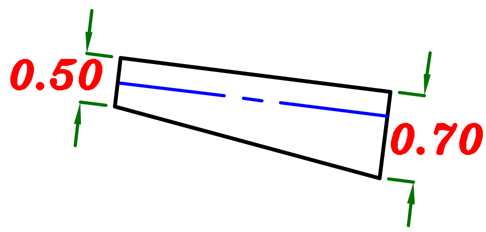
Check shear.



- **Allowable shear stress.**

- $q_{cu} = 0.24 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\frac{30}{1.5}} = 1.07 \ N\backslash mm^2$

- $q_{max.} = 0.7 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7 \sqrt{\frac{30}{1.5}} = 3.13 \ N\backslash mm^2$

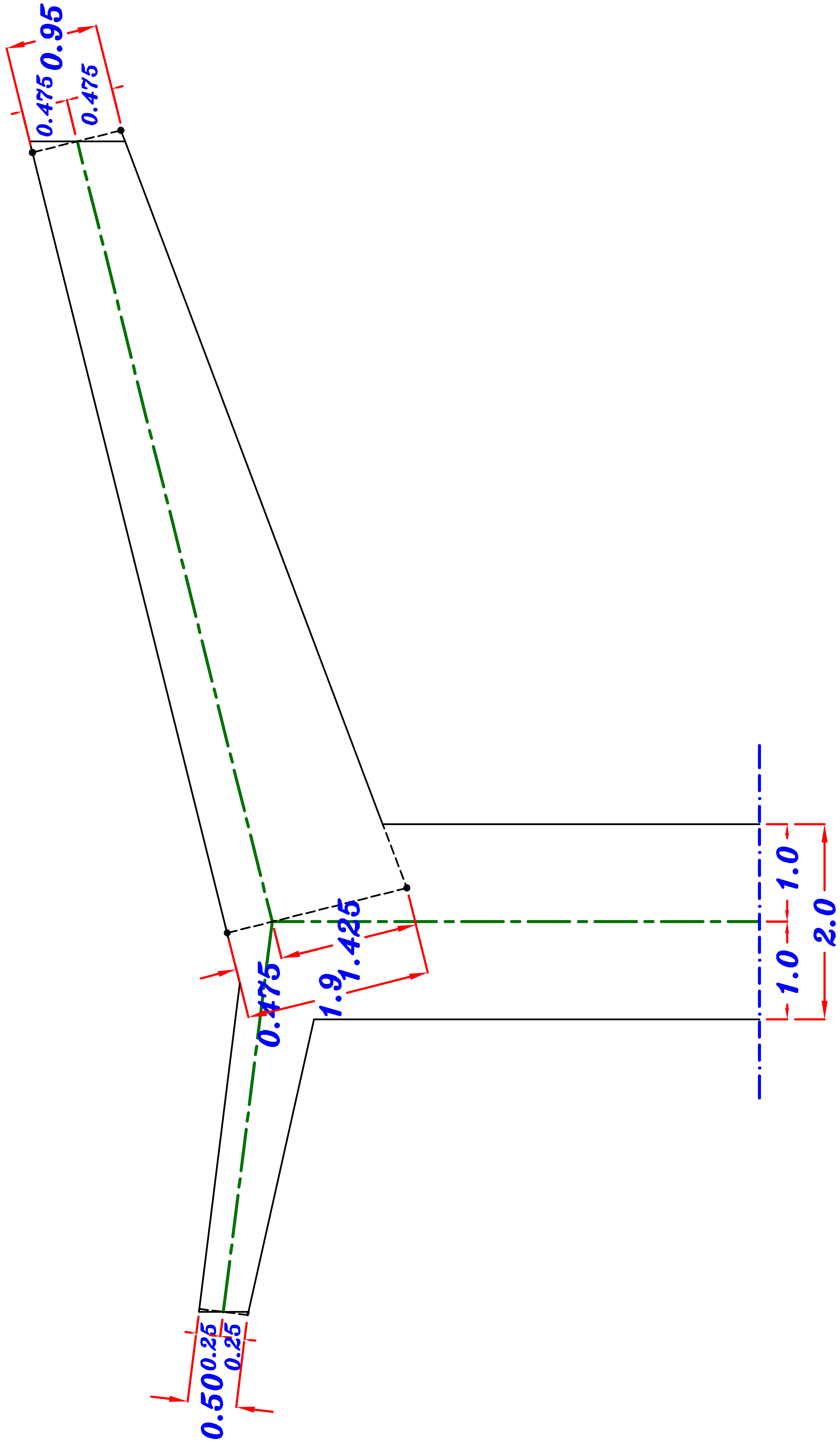


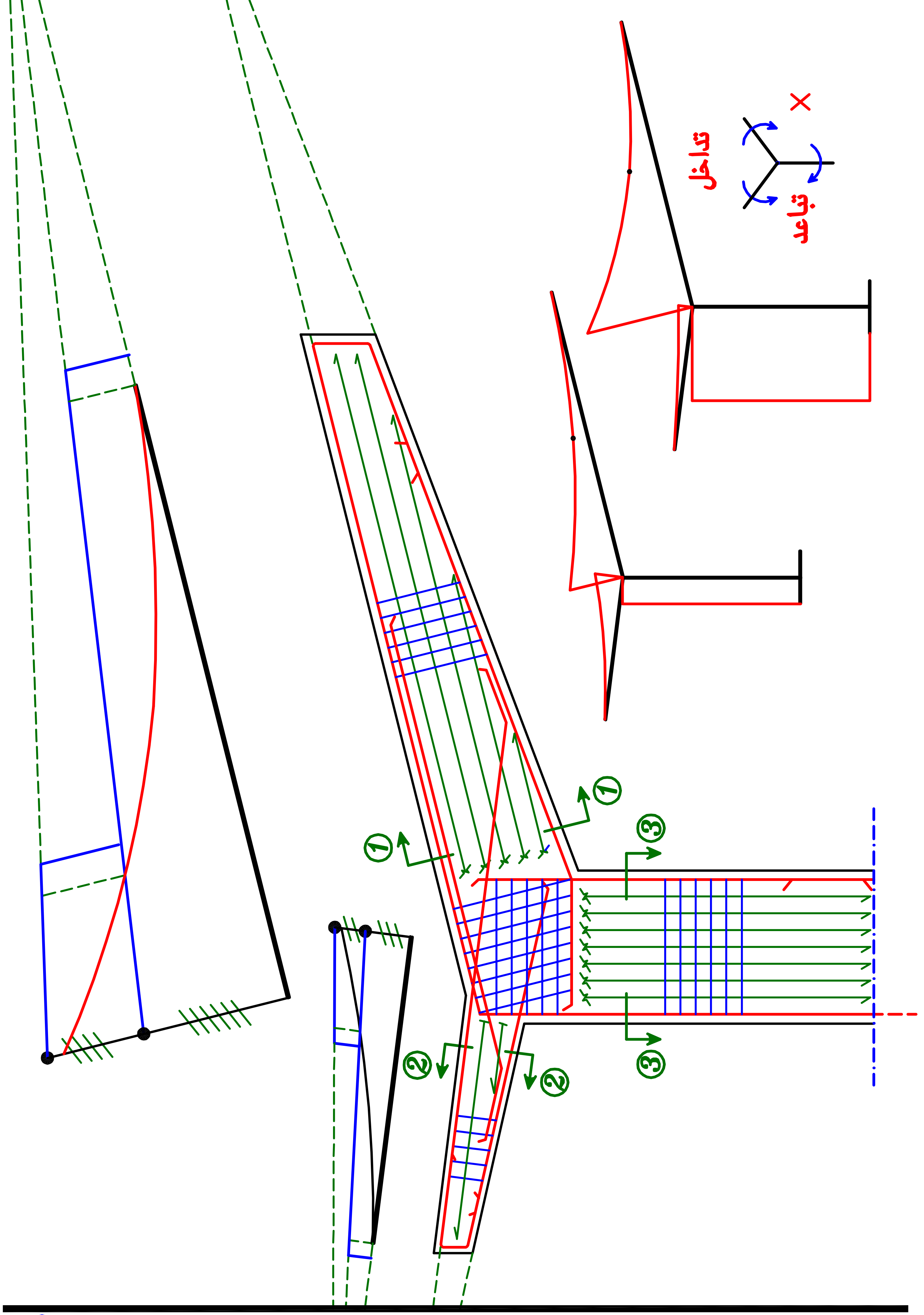
Sec. 1 $\quad Q = 110.35 \ kN$

$\therefore$ Actual shear stress. $= q_U = \frac{Q}{b \ d} - \frac{M \ \tan\beta}{b \ d^2}$

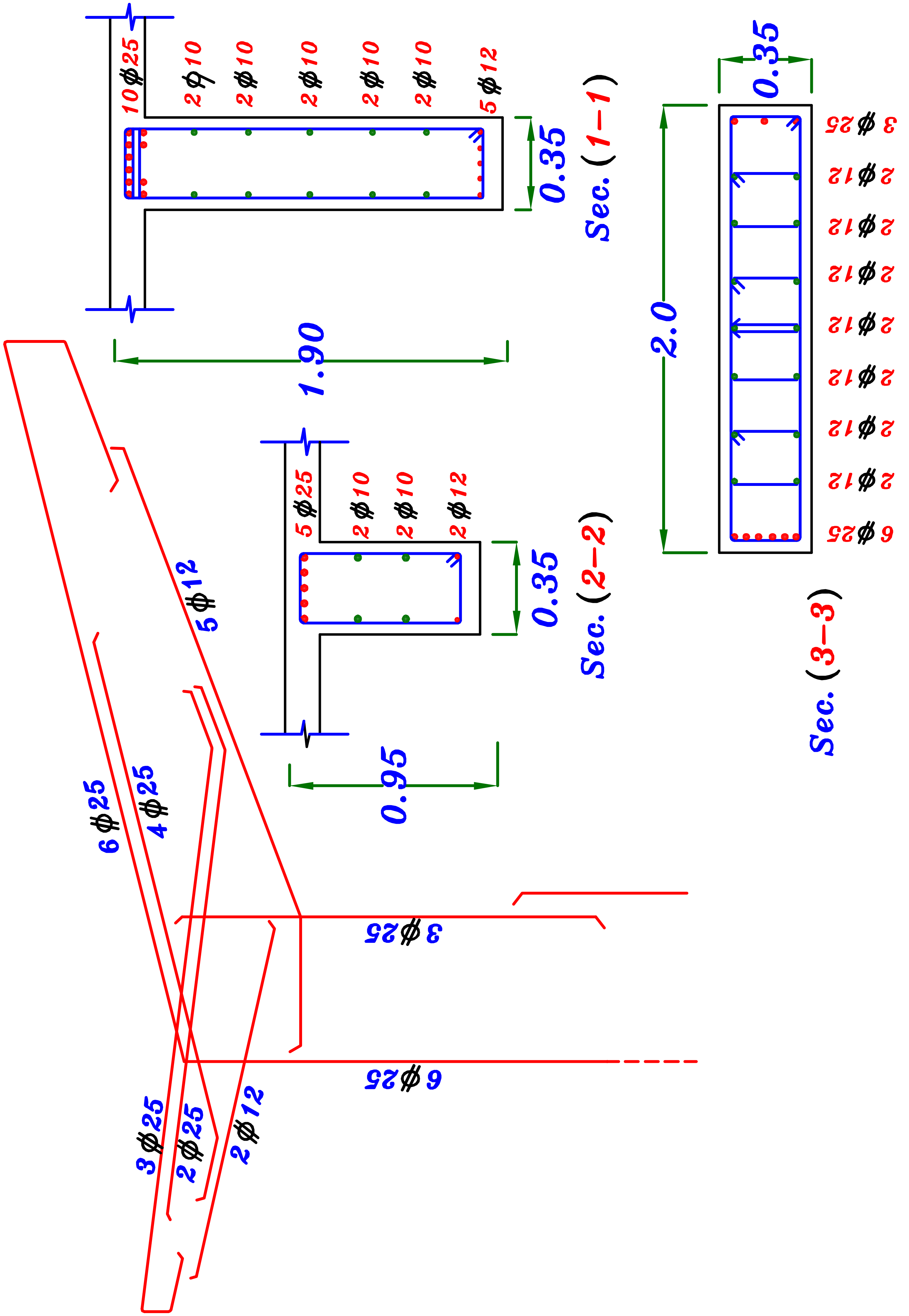
$q_U = \frac{110.35 * 10^3}{350 * 450} - ZERO = 0.70 \ N\backslash mm^2$

$\therefore q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5 \phi 8 \backslash m`}$

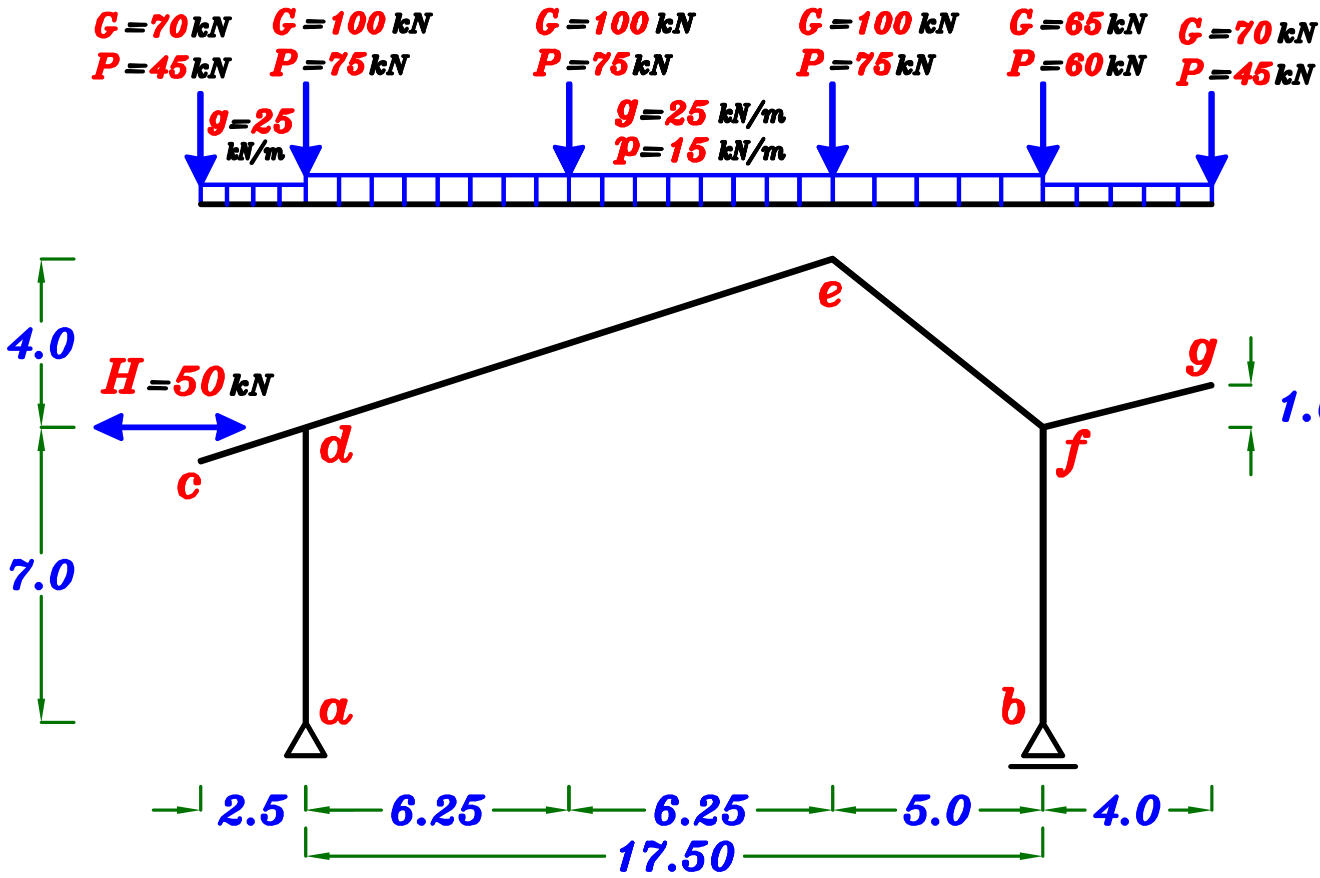

0.95
0.475
0.475
1.425
1.9
0.475
1.0
1.0
2.0
0.50
0.25
0.25



تداخل
تباعد
X
①
①
②
②
③
③


Sec. (1–1)
10 ϕ25
2 ϕ10
2 ϕ10
2 ϕ10
2 ϕ10
2 ϕ10
5 ϕ12
0.35
1.90
Sec. (2–2)
5 ϕ25
2 ϕ10
2 ϕ10
2 ϕ12
0.35
0.95
Sec. (3–3)
0.35
2.0
3 ϕ25
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
2 ϕ12
6 ϕ25
5 ϕ12
6 ϕ25
4 ϕ25
3 ϕ25
6 ϕ25
3 ϕ25
2 ϕ25
2 ϕ12

Example.



The Figure shows a statical system and load diagram For a reinforced concrete shed. The shed is covered with reinforced concrete slabs supported on a system of secondary beams and Frames (F), spaced at 6.0 m. Each Frame is subjected also to a horizontal wind (H) as shown on the load diagram. For an intermediate panel.

It is required to:

1- Draw the absolute B.M.D considering the directions of wind load (H). of an intermediate Frame (F), using given working loads.

2- Draw the N.F.D and S.F.D For case of total load only, neglecting the wind load (H).

3- Design the critical sections (at least Four sections) For the intermediate Frame (F). to satisfy both bending moments and normal Forces.

4- Sheck shear stresses at joint (f).

5- Consider the effect of buckling condition in the design of column (a-d).

6- Draw the details of reinforcement For the Frame, considering the moment of resistance principle For girder part (c-d-e-f-g) in elevation (to scale 1:50) and cross section (to scale 1:20).

Design Data:

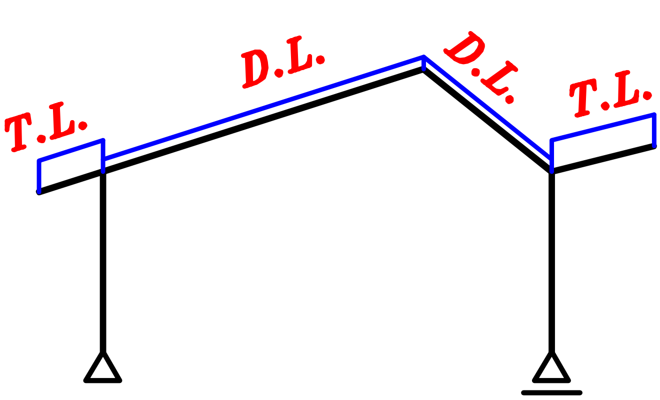
t_s = 140 mm b (beams) = 250 mm b (Frame) = 350 mm

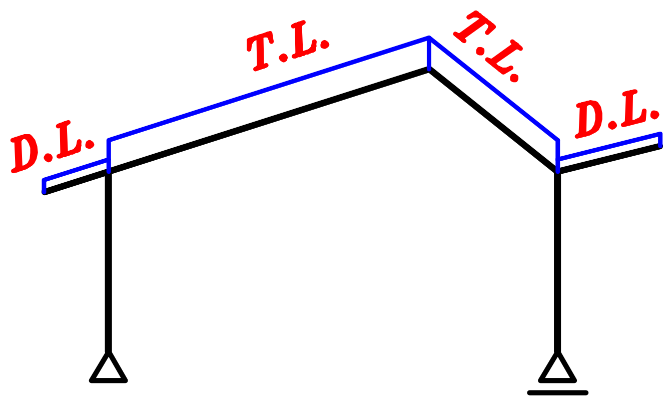
Spacing between Frames = 6.0 m

C 30 & Steel 360/520

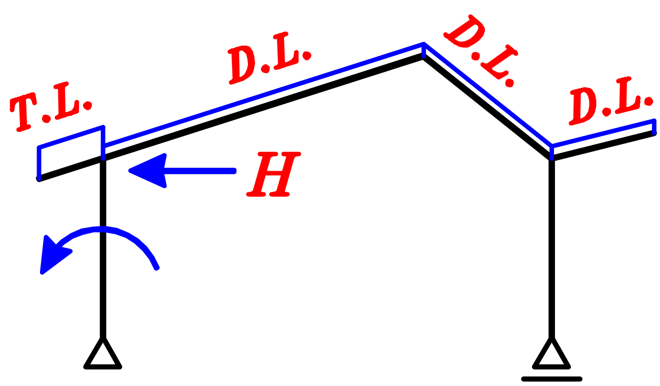
1 - Draw the absolute B.M.D considering the directions of wind load (H). of an intermediate Frame (F), using given working loads.

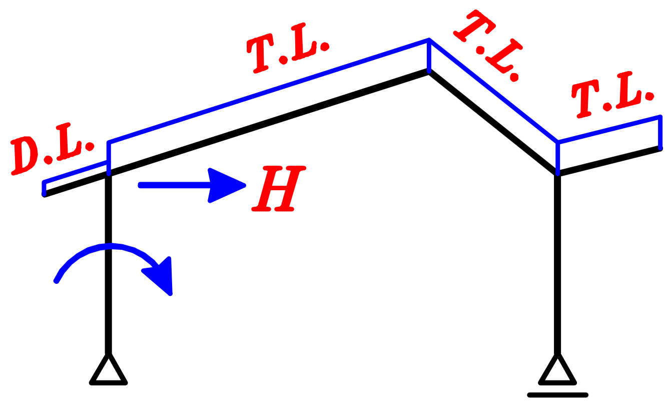
حالات التحميل الاساسيه لل Frame



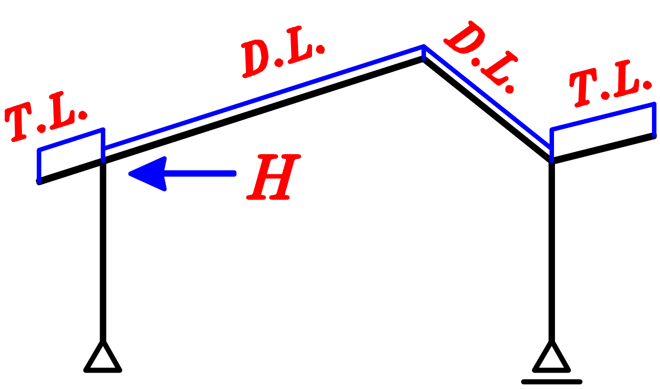


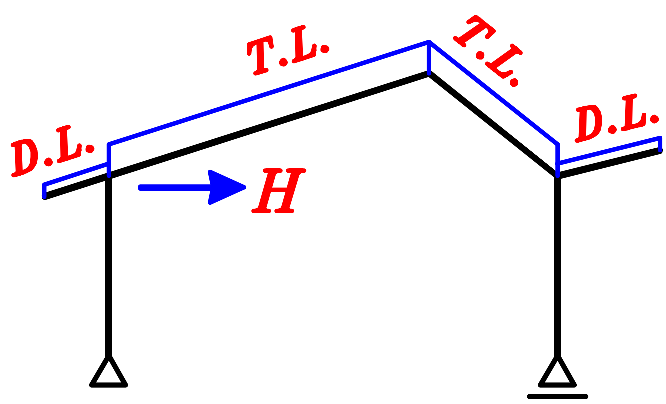
مع وجود القوى الافقيه **(H)** سنحتاج لحالتى تحميل أخريتين

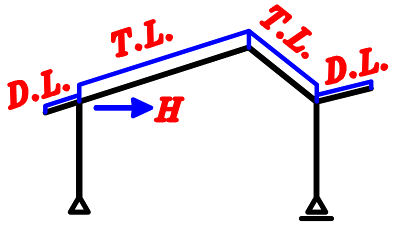


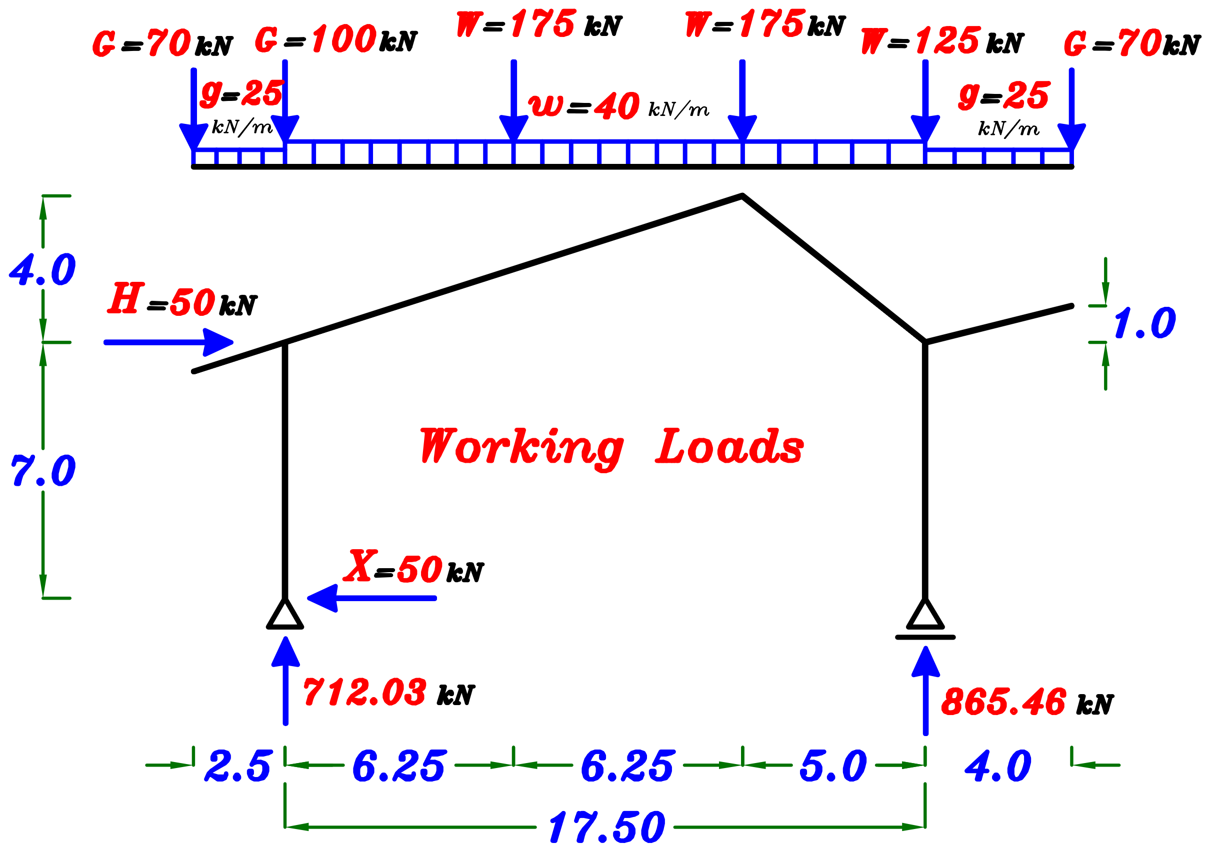


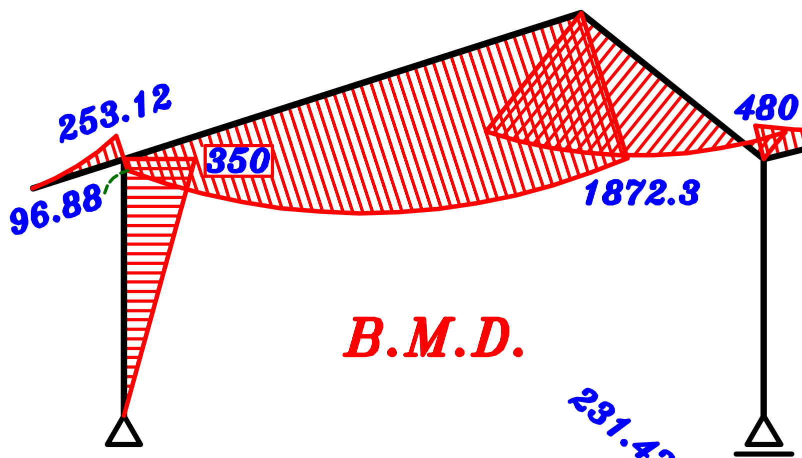
و نظرا لضيق الوقت سنأخذ حالات التحميل الاساسيه مع الاخذ فى الاعتبار القوه الافقيه

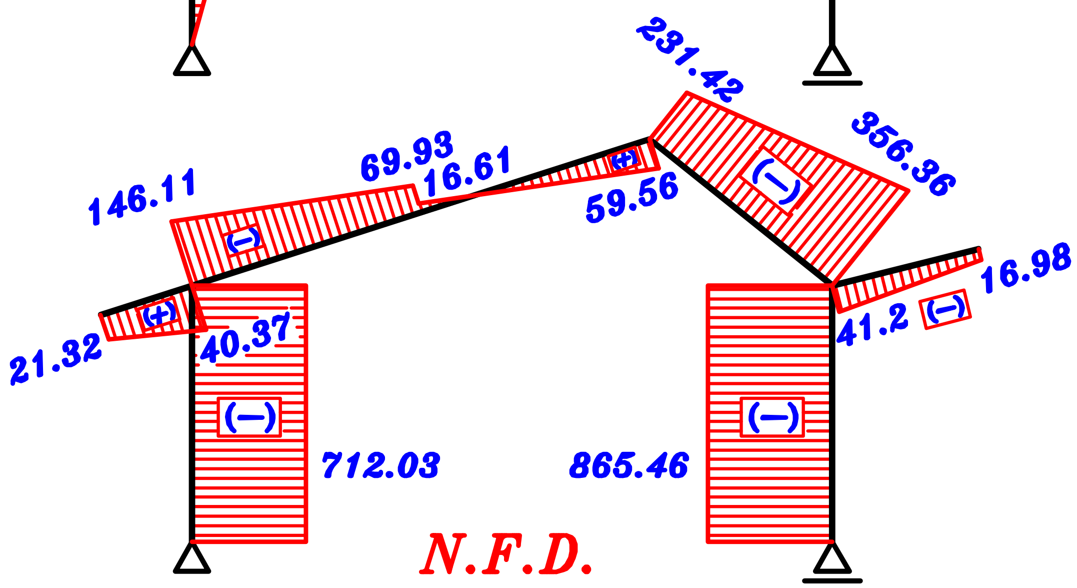


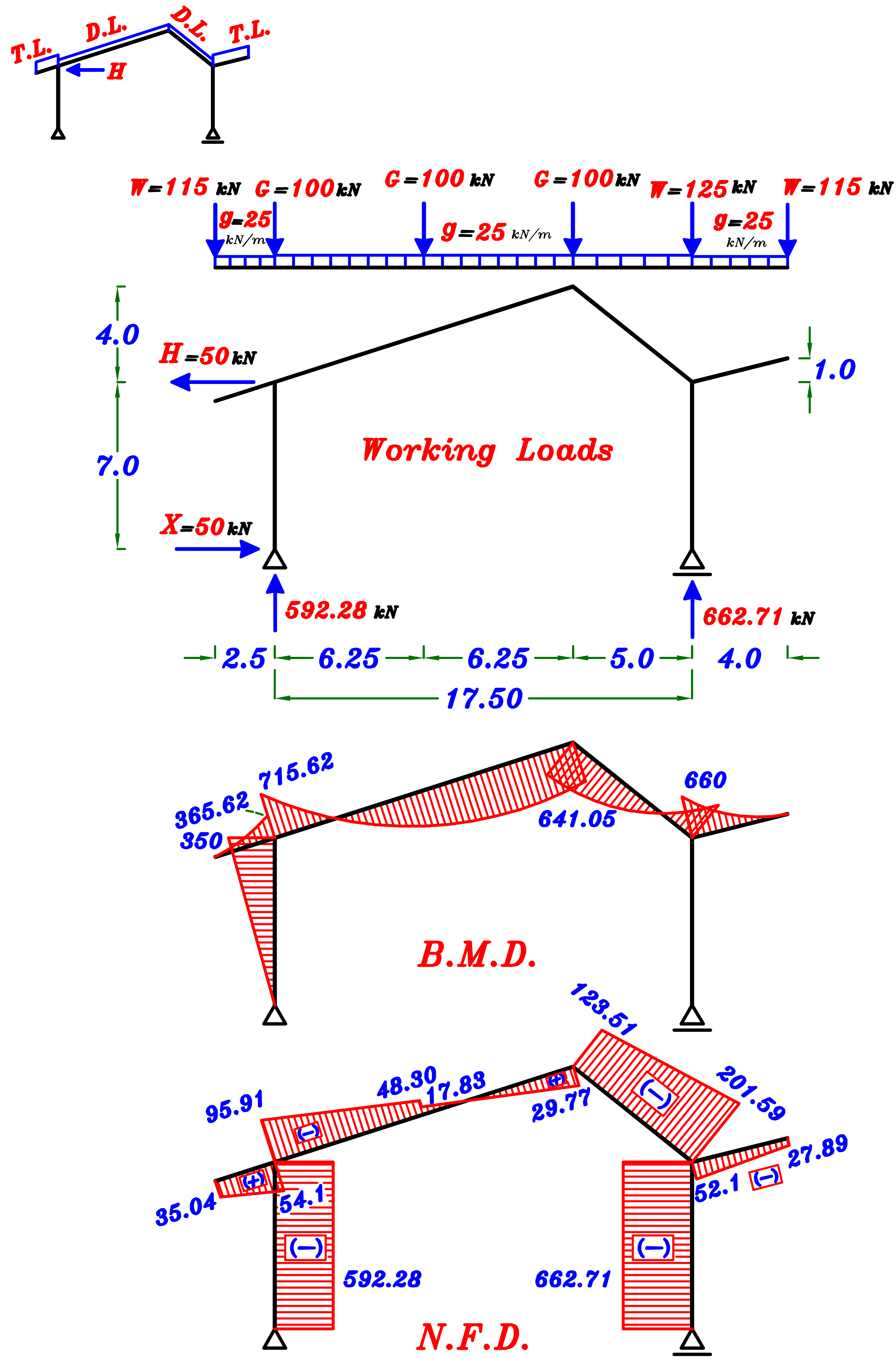



D.L.
T.L.
T.L.
D.L.
H

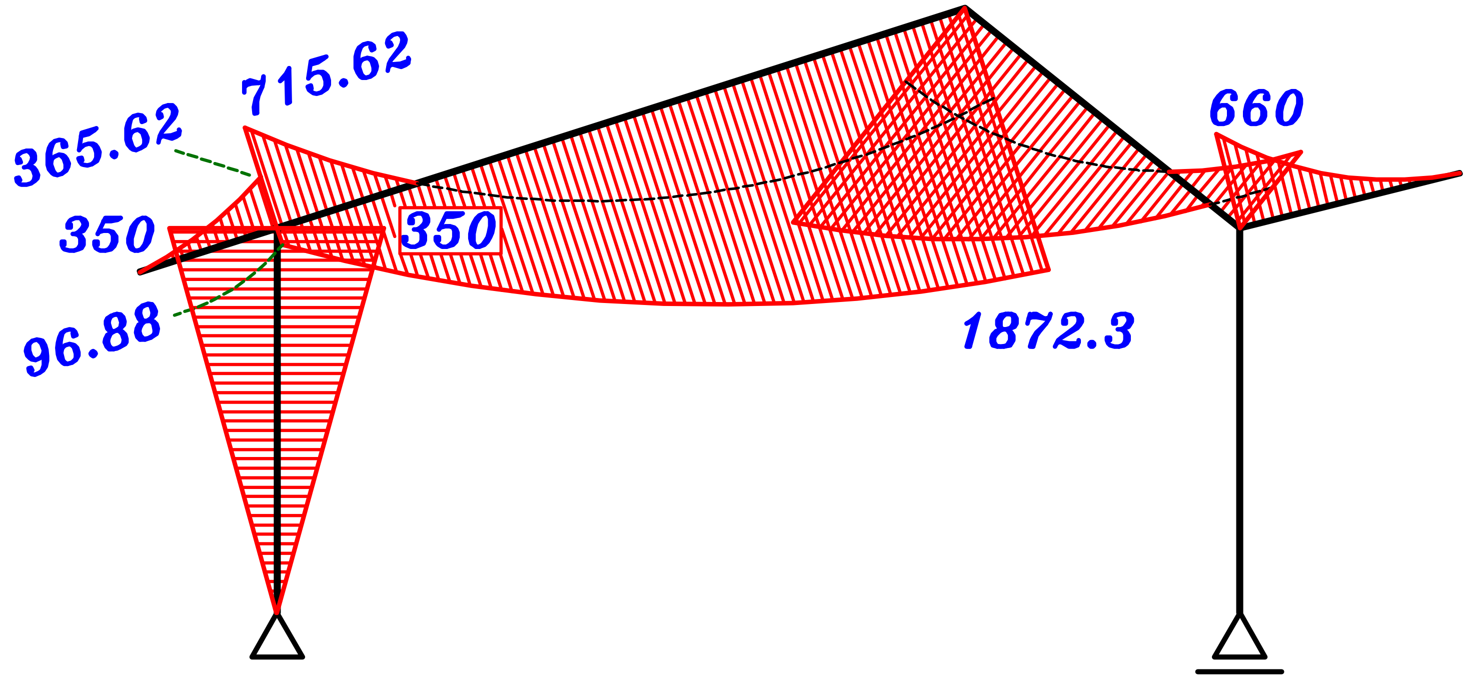

G=70 kN
G=100 kN
W=175 kN
W=175 kN
W=125 kN
G=70 kN
g=25 kN/m
w=40 kN/m
g=25 kN/m
4.0
H=50 kN
1.0
Working Loads
7.0
X=50 kN
712.03 kN
865.46 kN
2.5
6.25
6.25
5.0
4.0
17.50


253.12
350
480
96.88
1872.3
B.M.D.

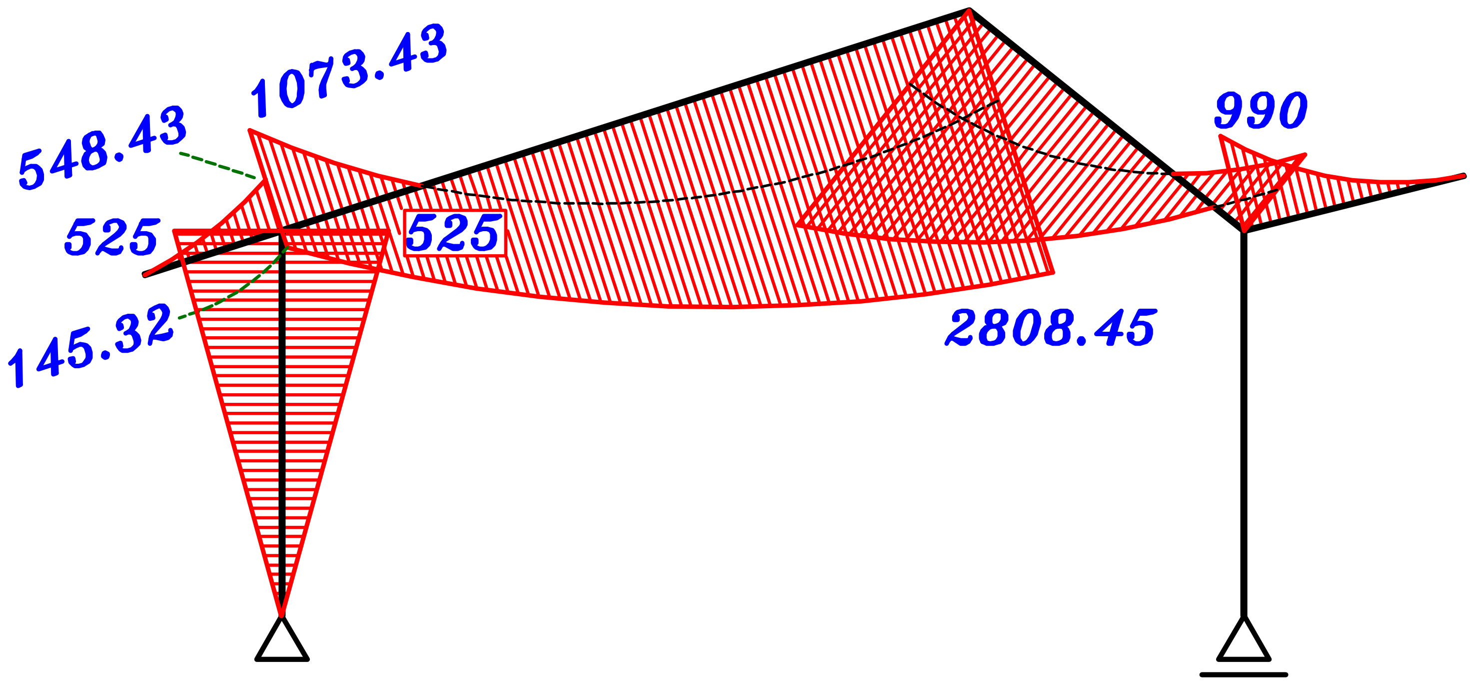

231.42
356.36
69.93
16.61
146.11
59.56
(+)
(−)
(−)
(+)
16.98
21.32
40.37
41.2
(−)
(−)
(−)
712.03
865.46
N.F.D.


T.L.
D.L.
D.L.
T.L.
H
W=115 kN
G=100 kN
G=100 kN
G=100 kN
W=125 kN
W=115 kN
g=25 kN/m
g=25 kN/m
g=25 kN/m
4.0
7.0
1.0
H=50 kN
X=50 kN
Working Loads
592.28 kN
662.71 kN
2.5
6.25
6.25
5.0
4.0
17.50
715.62
365.62
350
641.05
660
B.M.D.
123.51
201.59
95.91
48.30
17.83
29.77
35.04
54.1
52.1
27.89
592.28
662.71
N.F.D.

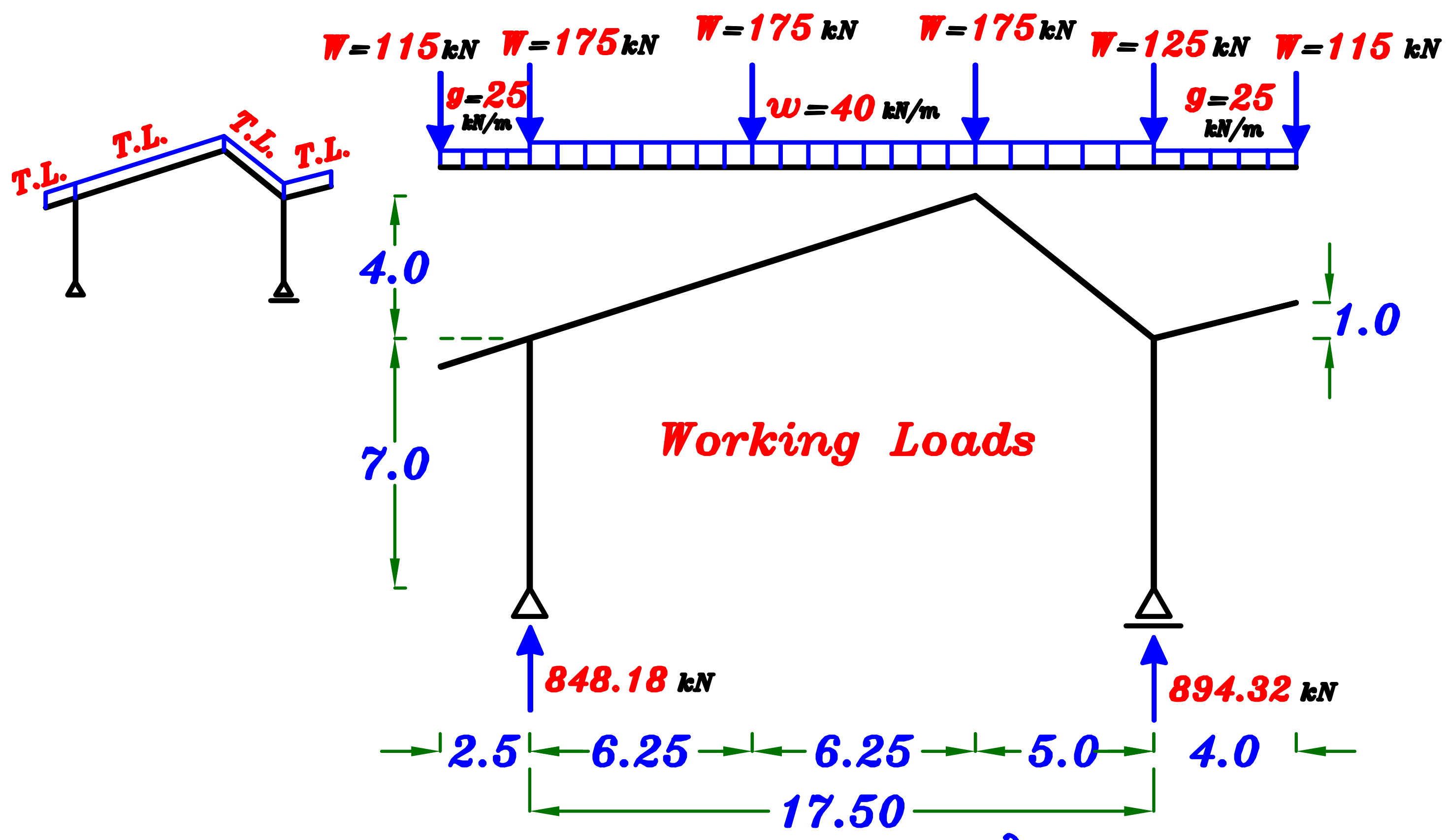
max.–max. B.M.D. working Loads

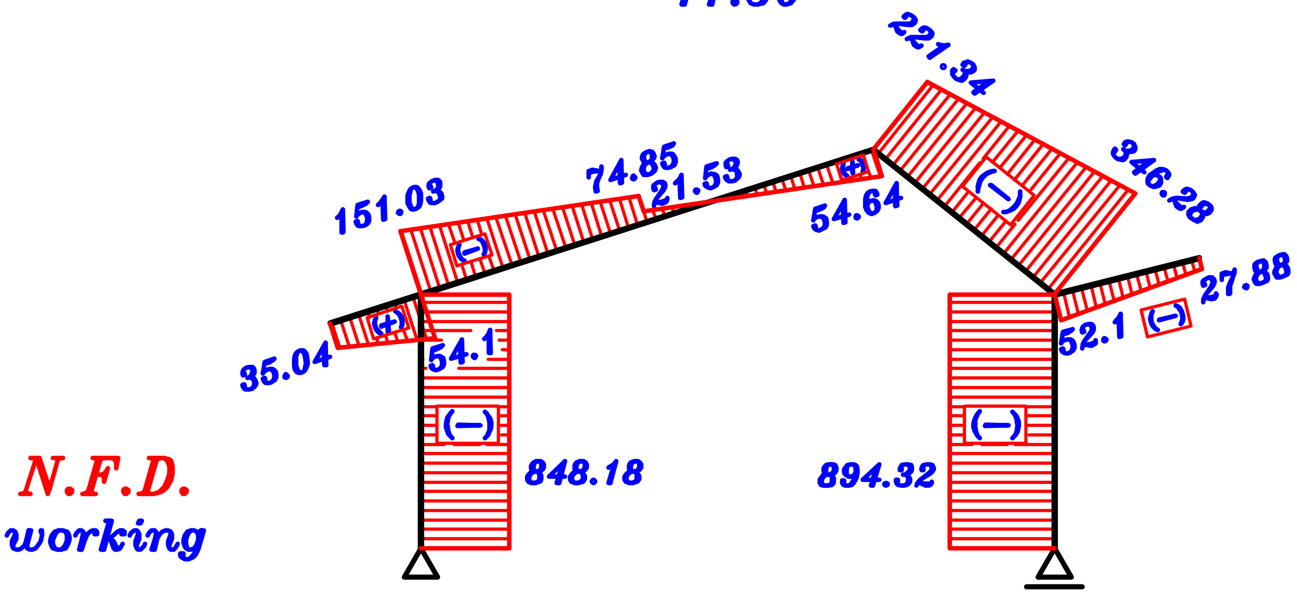


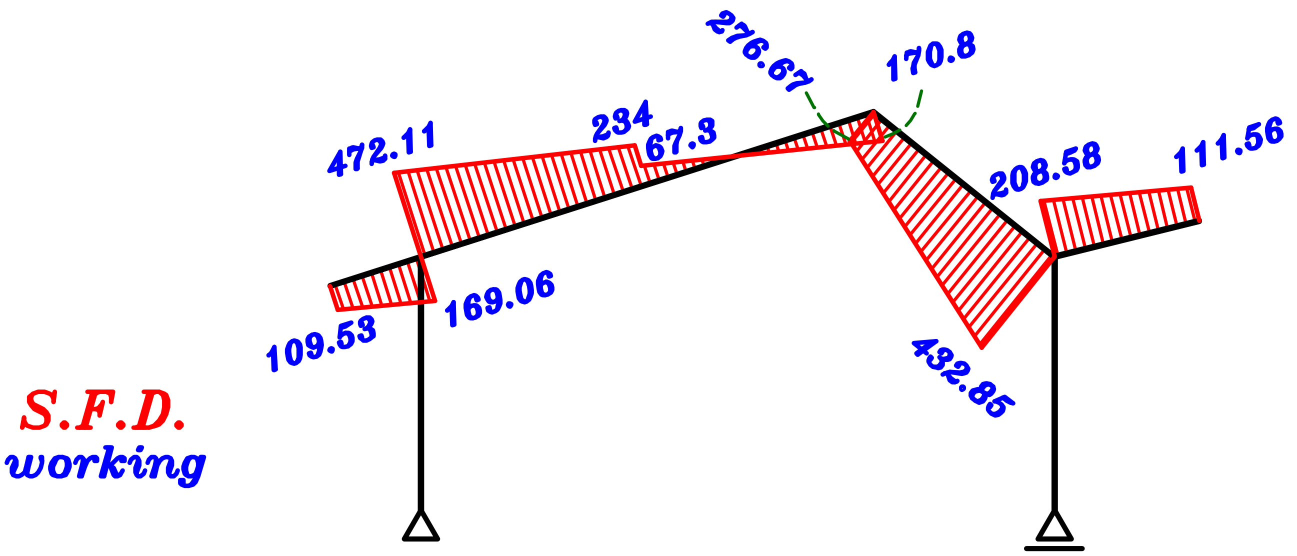
max.–max. B.M.D. U.L.



2- Draw the N.F.D and S.F.D For case of total load only, neglecting the wind load (H).

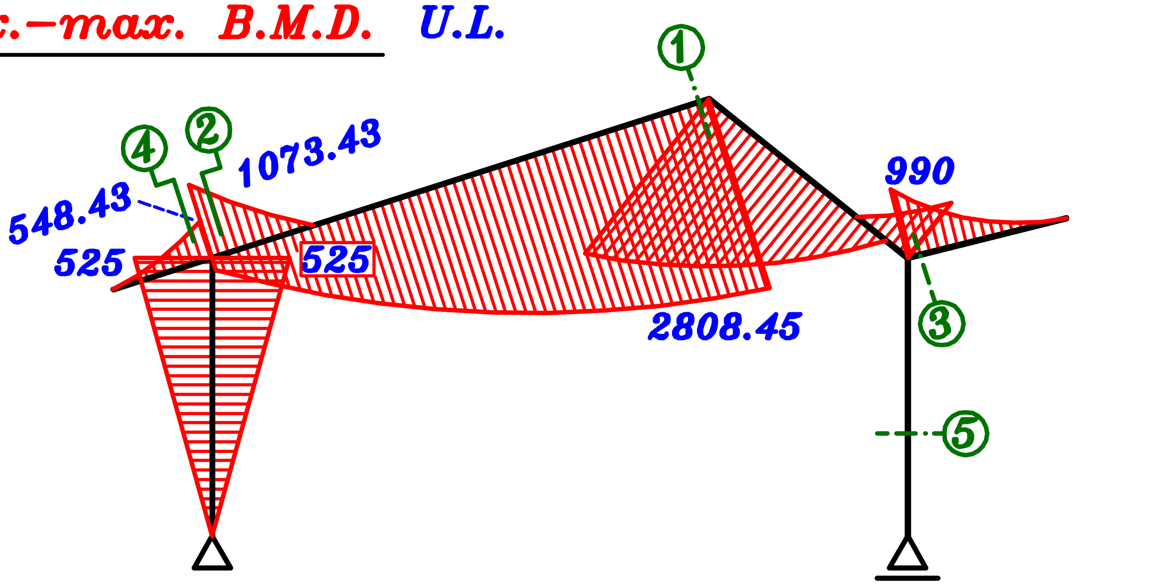






3- Design the critical sections (at least Four sections) For the intermediate Frame (F).
to satisfy both bending moments and normal Forces.

max.-max. B.M.D. U.L.



Sec. 1

$M = 2808.45$ kN.m , $T = 59.56 * 1.5 = 89.34$ kN , $b = 350$ mm

$$d = 3.5\sqrt{\frac{2808.45*10^6}{30*350}} = 1810.1 \text{ mm} \quad (\text{as R-Sec.})$$

$\therefore$ Take $\boxed{d = 1900 \text{ mm}}$, $\boxed{t = 2000 \text{ mm}}$

$$e = \frac{M}{T} = \frac{2808.45}{89.34} = 31.43 \text{ m} \quad \therefore \frac{e}{t} = \frac{31.43}{2.0} = 15.71 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e - \frac{t}{2} + C = 31.43 - \frac{2.0}{2} + 0.10 = 30.53 \text{ m}$$

$$M_s = T * e_s = 89.34 * 30.53 = 2727.55 \text{ kN.m}$$

$$\therefore d = C_1\sqrt{\frac{M_s}{F_{cu}\, b}} \quad \therefore 1900 = C_1\sqrt{\frac{2727.55*10^6}{30*350}} \rightarrow C_1 = 3.73 \rightarrow J = 0.792$$

$$\therefore A_s = \frac{M_s}{J\, F_y\, d} + \frac{T_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{2727.55*10^6}{0.792*360*1900} + \frac{89.34*10^3}{(360 \backslash 1.15)} = 5320.3 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 5320.3 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1900 = 2276.5 \ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore Take \ A_s = A_{s_{req.}} = 5320.3 \ mm^2 \qquad 11 \phi 25$$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{25 + 25} = 6.50 = 6.0 \ bars$$

ملحوظه لان قيمه ال Tension صغيره بالنسبه لل moment
فاذا اهملنا ال Tension فى هذه الحاله و صممنا على ال moment فقط
فلن تفرق كثيرا فى كميه الحديد المطلوبه .

Sec. ②

$$M = 1073.43 \ kN.m \ , \ P = 95.91 * 1.5 = 143.86 \ kN$$

$$, \ b = 350 \ mm \ , \ t = 2000 \ mm$$

$$Check \quad \frac{P}{F_{cu}\, b\, t} = \frac{143.86 * 10^3}{30 * 350 * 2000} = 0.006 < 0.04 \quad \therefore (neglect \ P)$$

$$\because d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}} \quad \therefore 1900 = C_1 \sqrt{\frac{1073.43 * 10^6}{30 * 350}} \longrightarrow C_1 = 5.94 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{1073.43 * 10^6}{0.826 * 360 * 1900} = 1899.9 \ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1899.9 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1900 = 2276.5 \ mm^2$$

$$\therefore \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{Use} A_{s_{min.}}$$

$$\left.\begin{aligned} A_{s_{min.}} &= \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1900 = 2276.5 \ mm^2 \\ 1.3\, A_{s_{req.}} &= 1.3 * 1899.9 = 2470 \ mm^2 \end{aligned}\right\} = 2276.5 \ mm^2 \qquad 5 \phi 25$$

الأقل

Sec. ③

$M = 990\ kN.m\ ,\ P = 52.1 * 1.5 = 78.15\ kN\ ,\ b = 350\ mm$

$$d_o = 3.5\sqrt{\frac{990 * 10^6}{30 * 350}} = 1074.71\ mm \quad (as\ R\text{-}Sec.)$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1074.71) = (1182.18 \rightarrow 1397.12)\ mm$$

Take $d = 1200\ mm\ ,\ t = 1200 + 100 = 1300\ mm$

Check $$\frac{P}{F_{cu}\, b\, t} = \frac{78.15 * 10^3}{30 * 350 * 1300} = 0.0057 < 0.04 \quad \therefore (neglect\ P)$$

$\therefore$ Take $d = d_o = 1074.71\ mm$

$\therefore$ Take $\boxed{d = 1100\ mm}\ ,\ \boxed{t = 1200\ mm}$

$\because\ C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \frac{990 * 10^6}{0.780 * 360 * 1074.71} = 3280.55\ mm^2$$

Check $A_{s_{min.}}$

$$A_{s_{req.}} = 3280.55\ mm^2$$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1100 = 1317.9\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3280.55\ mm^2$ $7\phi 25$

Sec. ④

$M = 548.43\ kN.m\ ,\ T = 54.10 * 1.5 = 81.15\ kN$

$,\ b = 350\ mm\ ,\ t = 1200\ mm$ = The same depth of Sec. ③

$$e = \frac{M}{T} = \frac{548.43}{81.15} = 6.758\ m$$

$$\therefore \frac{e}{t} = \frac{6.758}{1.2} = 5.63 > 0.5 \xrightarrow{Use} e_s$$

$$e_s = e - \frac{t}{2} + c = 6.758 - \frac{1.20}{2} + 0.10 = 6.258 \; m$$

$$M_s = T * e_s = 81.15 * 6.258 = 507.83 \; kN.m$$

$$\therefore 1100 = C_1 \sqrt{\frac{M_s}{F_{cu}\, b}} = C_1 \sqrt{\frac{507.83 * 10^6}{30 * 350}} \rightarrow C_1 = 5.0 \rightarrow J = 0.826$$

$$\therefore A_s = \frac{M_s}{J\, F_y\, d} + \frac{T_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{507.83 * 10^6}{0.826 * 360 * 1100} + \frac{81.15 * 10^3}{(360 \backslash 1.15)} = 1811.7 \; mm^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 1811.7 \; mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{30}}{360}\right) 350 * 1100 = 1317.9 \, mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1811.7 \; mm^2$ $\quad 4 \phi 25$

Sec. 5 $\quad (350 * 1000)$

Axially Loaded Column. $\quad P = 894.32 * 1.5 = 1341.48 \; kN$

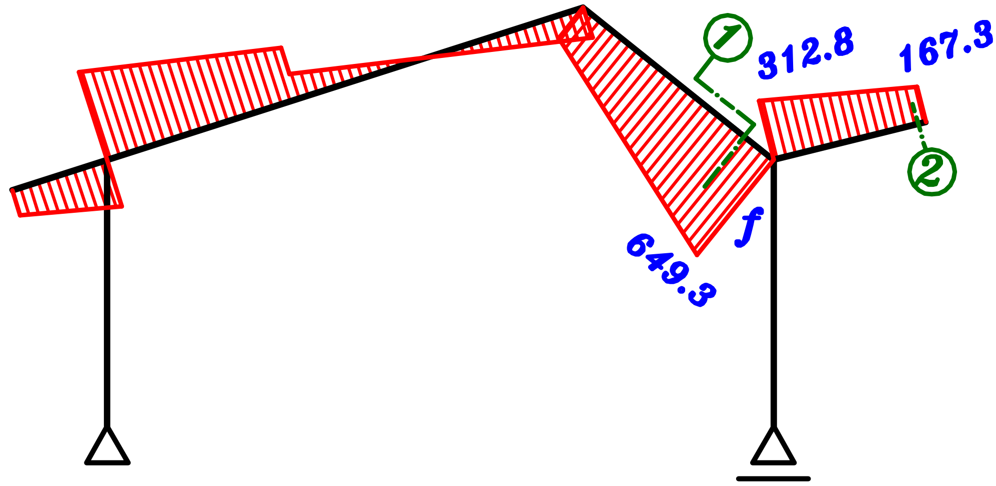
$$\therefore P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$$

$$\therefore 1341.48 * 10^3 = 0.35\, (350 * 1000)(30) + 0.67\, A_s\, (360)$$

$\therefore A_s = -9674 \; mm^2 = (-Ve)$ Value

$$\therefore A_s = A_{s_{min.}} = \frac{0.8}{100} * 350 * 1000 = 2800 \; mm^2$$ $\quad 12 \phi 18$

4- Sheck shear stresses at joint (f)



S.F.D.
U.L.

- **Allowable shear stress.**

- $q_{cu} = 0.24 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.24 \sqrt{\dfrac{30}{1.5}} = 1.07 \ N\backslash mm^2$

- $q_{max.} = 0.7 \sqrt{\dfrac{F_{cu}}{\gamma_c}} = 0.7 \sqrt{\dfrac{30}{1.5}} = 3.0 \ N\backslash mm^2$

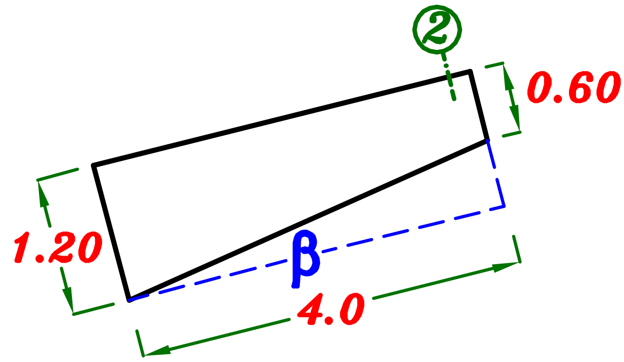
Sec. 1 $Q = 649.3$ kN $b = 350$ mm , $b = 1900$ mm

$\therefore \ q_U = \dfrac{Q}{b\,d} = \dfrac{649.3 * 10^3}{350 * 1900} = 0.97 \ N\backslash mm^2$

$\therefore \ q_{cu} > q_U$ $\therefore$ Use min. Stirrups $\boxed{5 \phi 8 \backslash m`}$ 2 b

Sec. 2

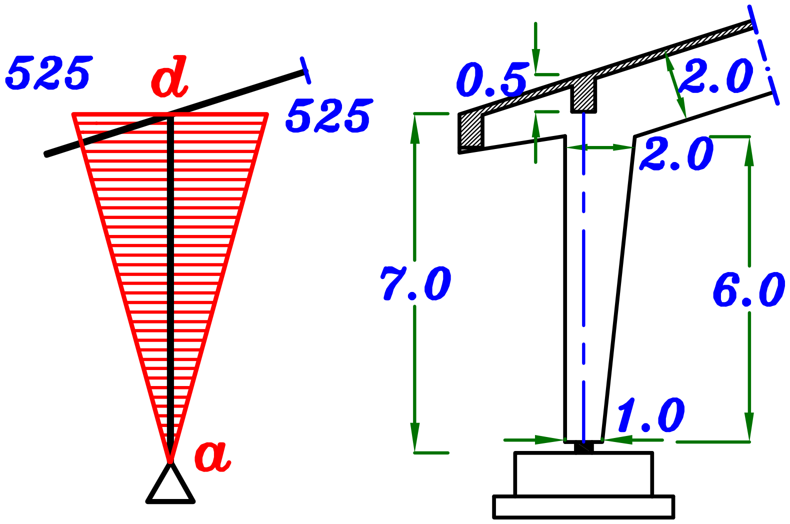
$$\boxed{q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\,d^2}}$$



$q_U = \dfrac{167.3 * 10^3}{350 * 550} - \text{zero} = 0.87 \ N\backslash mm^2$

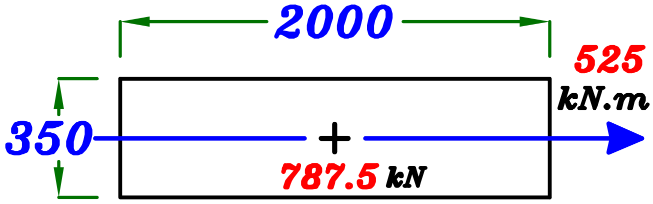
$\therefore \ q_{cu} > q_U$ $\therefore$ Use min. Stirrups $\boxed{5 \phi 8 \backslash m`}$ 2 b

5- Consider the effect of buckling condition in the design of column (a-d).



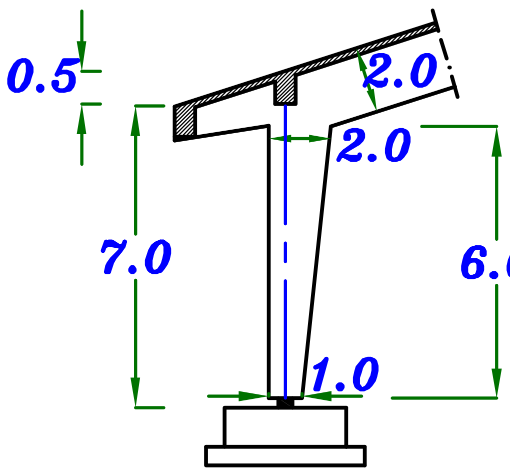
$M_{ext.} = 525$ kN.m

$P = 525 * 1.5 = 787.5$ kN



Check Buckling.

① In plane.



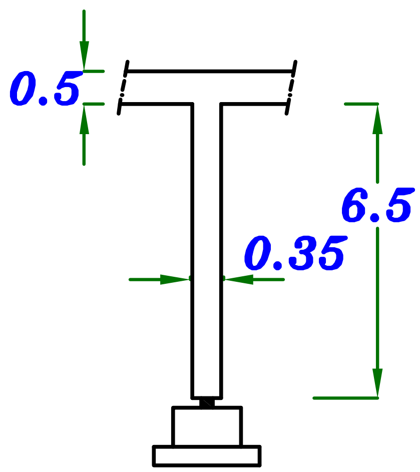
Upper Case ①
Lower Case ③ } k = 1.6

$H_o = 6.0$ m

$$\lambda_{b_{in}} = \frac{1.6 * 6.0}{2.0} = 4.80 < 10$$

Short Col.

② Out of plane.



Upper Case ①
Lower Case ③ } k = 1.6

$H_o = 6.50$ m

$$\lambda_{b_{out}} = \frac{1.6 * 6.5}{0.35} = 29.7 > 23$$

Unsafe Buckling

Increase b ⟶ b = 0.50 m

$$\lambda_{b_{out}} = \frac{1.6 * 6.5}{0.50} = 20.8 > 10$$

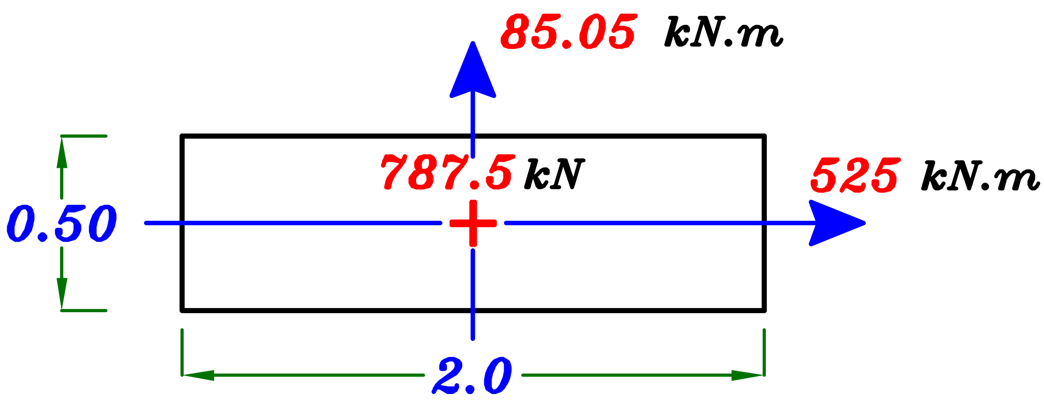
Long Col.

Take the bigger value of $\lambda_b = 20.8$ (Out of plane)

The Buckling Out of plane.

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{20.8^2 * 0.50}{2000} = 0.108 \quad m$$

$$M_{add.} = P * \delta = 787.5 * 0.108 = 85.05 \ kN.m$$



Using Bi-axial I.D.

assume $\zeta = 0.90$ --------- ECCS لا توجد قيمه غيرها فى ال

$$R_b = \frac{P}{F_{cu}\, b\, t} = \frac{787.5 * 10^3}{30 * 500 * 2000} = 0.026 \longrightarrow R_b = 0.30$$

ECCS Design Aids Page 5-13

$$\left.\begin{array}{l} \dfrac{M_X}{F_{cu}\, b\, t^2} = \dfrac{85.05 * 10^6}{30 * 2000 * 500^2} = 0.0056 \\ \dfrac{M_Y}{F_{cu}\, t\, b^2} = \dfrac{525.0 * 10^6}{30 * 500 * 2000^2} = 0.0087 \end{array}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 0.003$$

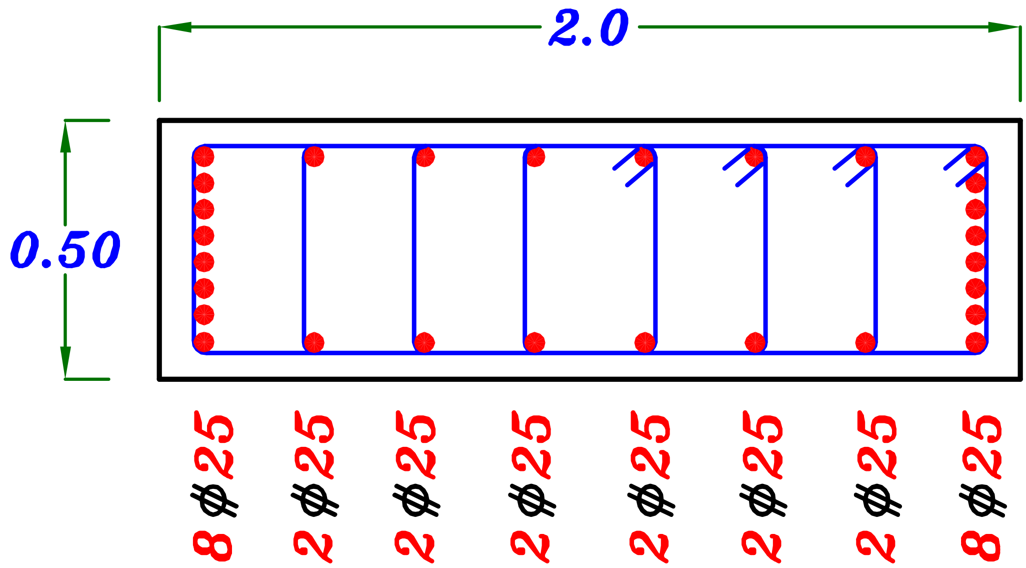
$$A_{s\,total} = \mu * b * t = 0.003 * 500 * 2000 = 3000 \ mm^2$$

$$A_{s_{min}} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

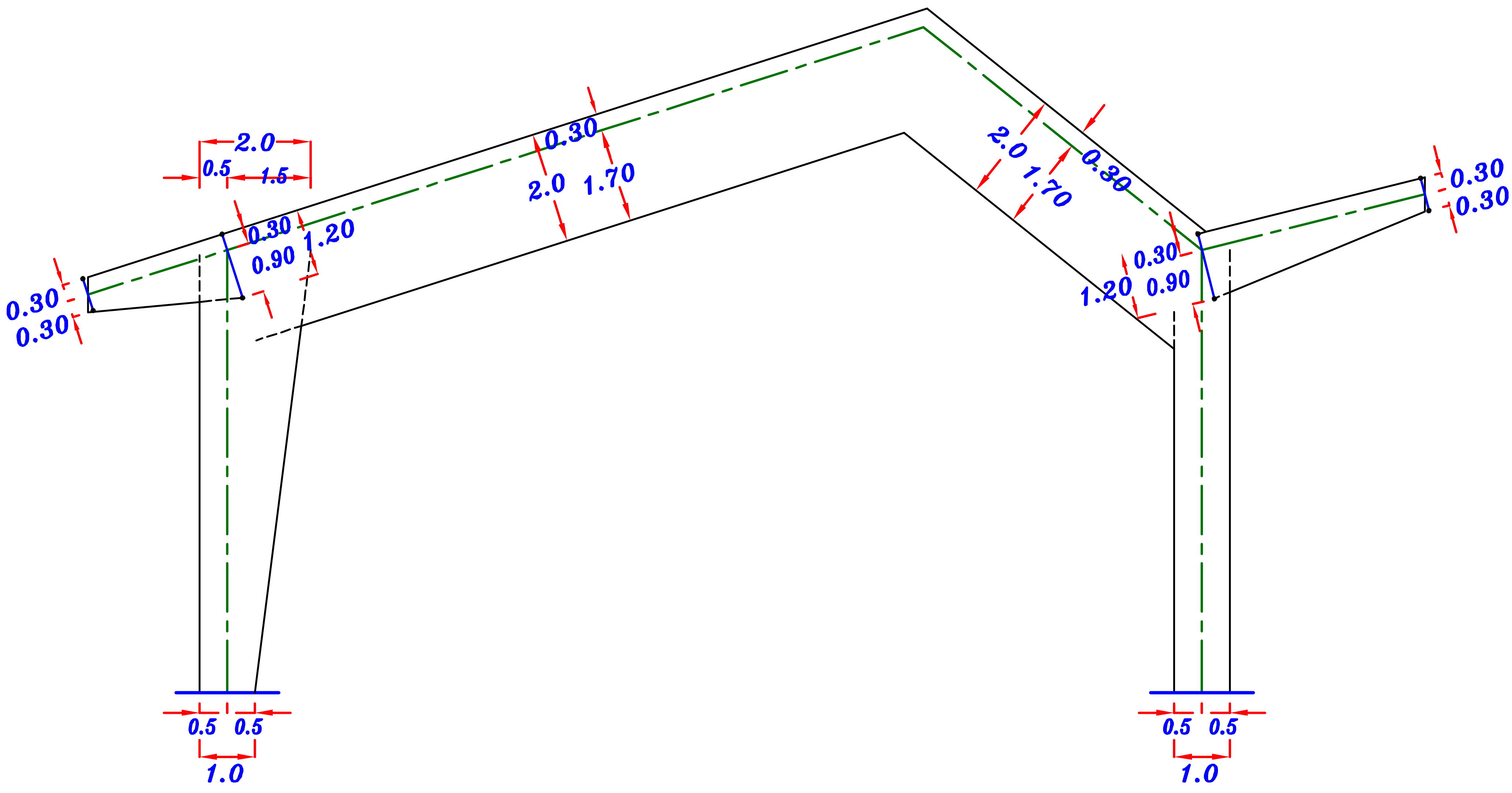
$$= \frac{0.25 + 0.052\,(20.8)}{100} * 500 * 2000 = 13316\ mm^2 > A_{s_{total}}$$

$Take\ A_s = A_{s_{min}} = 13316\ mm^2$ 28 Ø 25

توزع ٤ أسياخ فى الاركان و الباقى يوزع على الاربعه جهات .

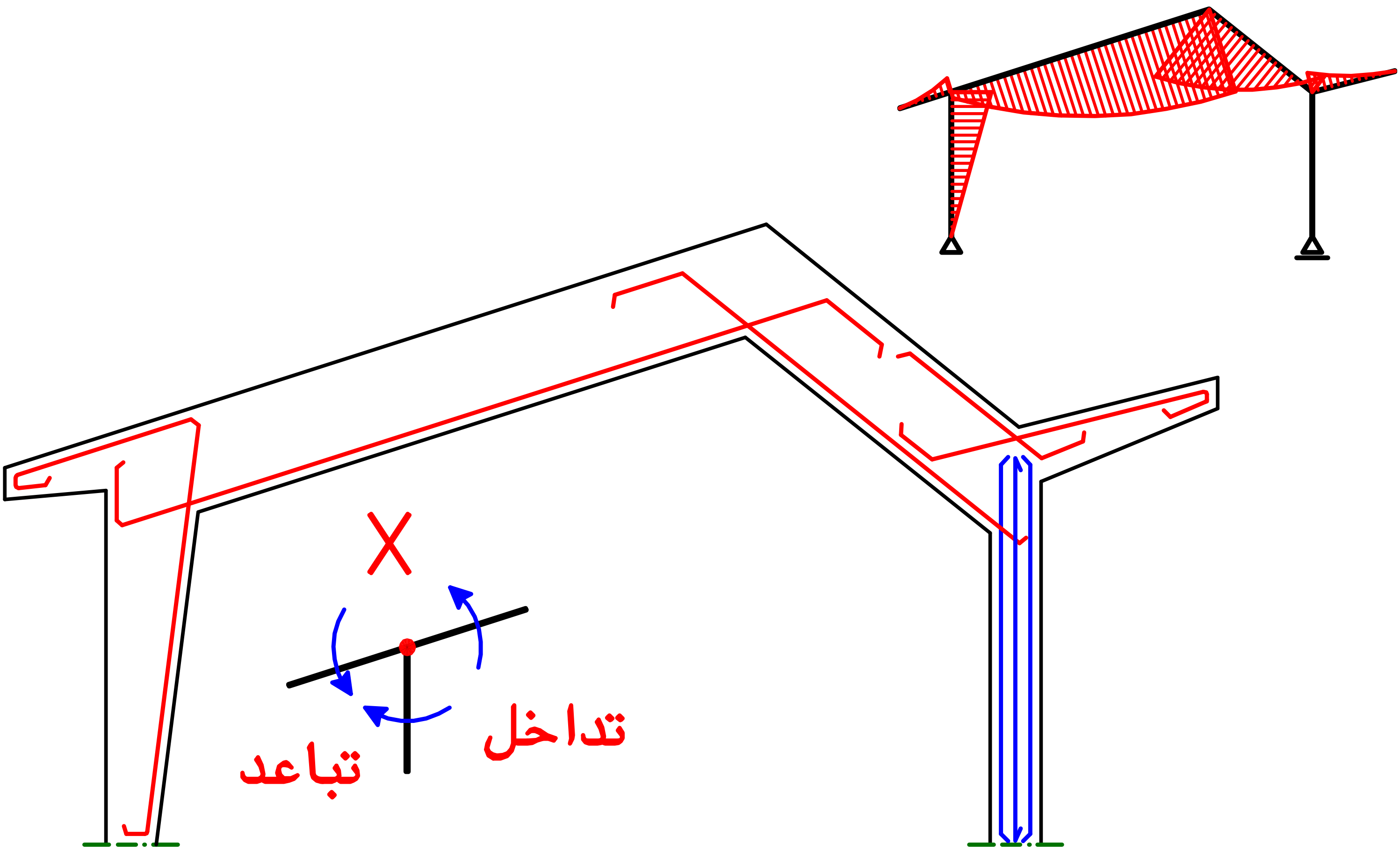


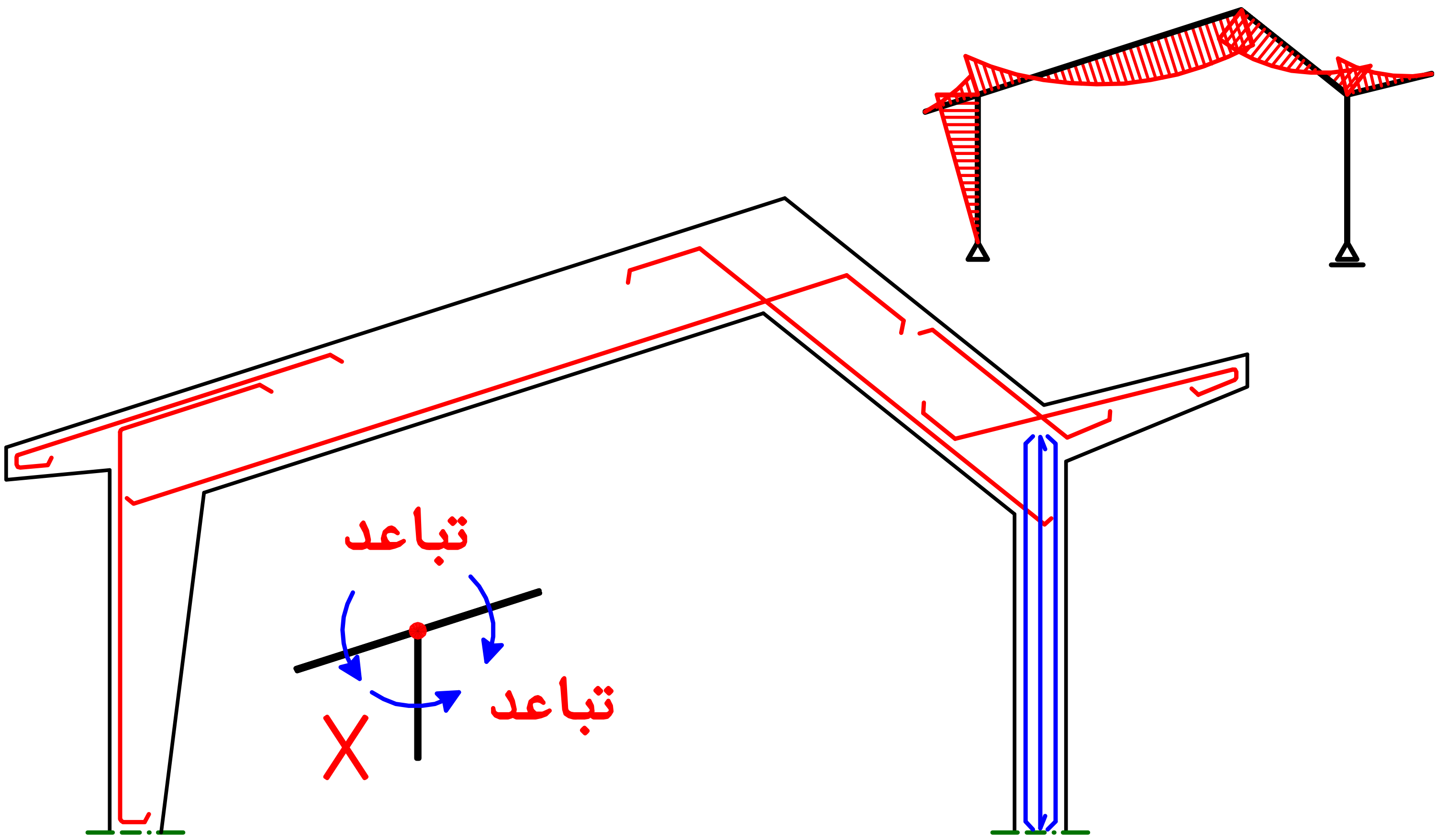
6- Draw the details of reinforcement For the Frame, considering **the moment of resistance principle** For girder part (c-d-e-f-g) in elevation (to scale 1:50) and cross section (to scale 1:20).

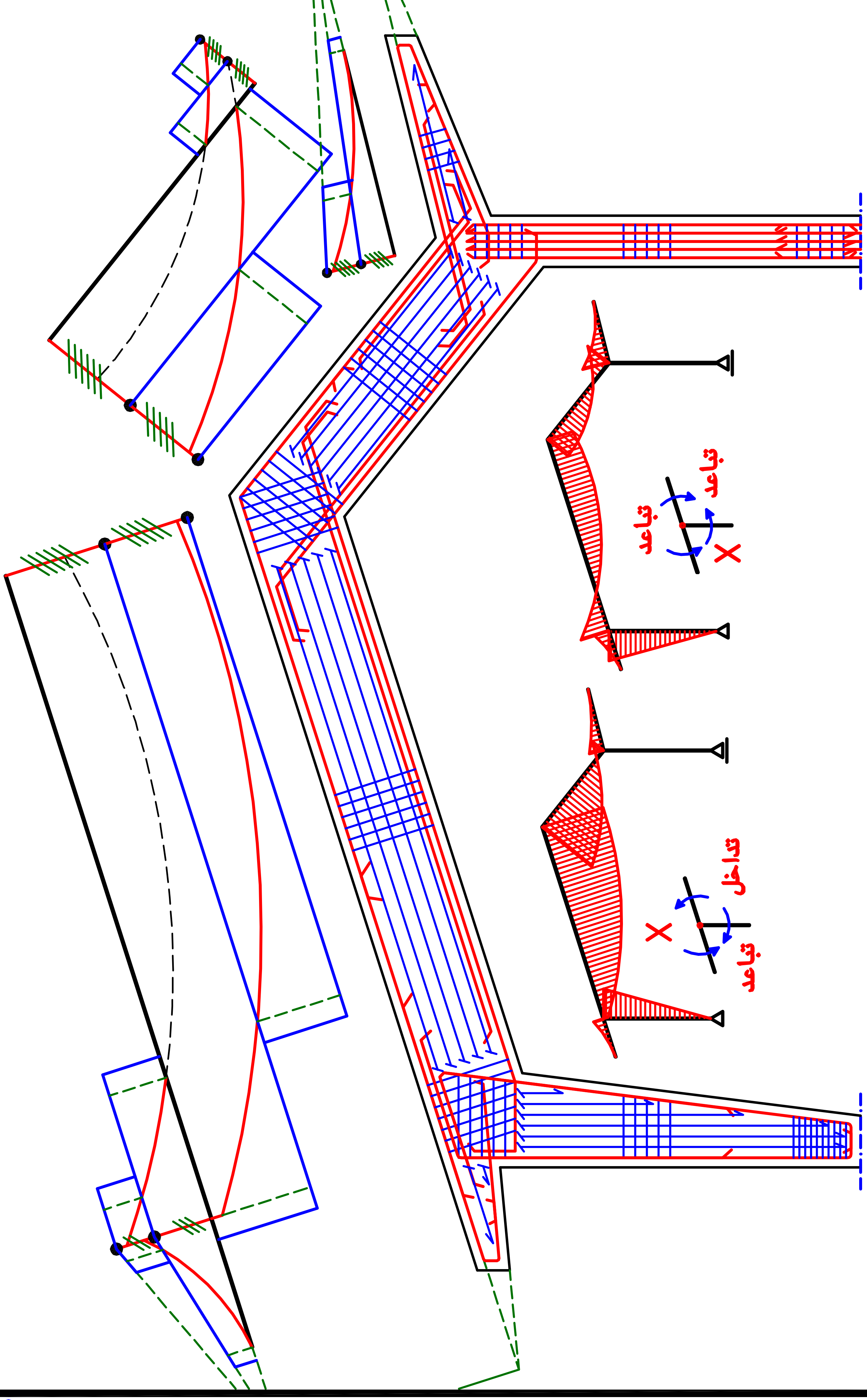


لا نرسم التسليح على شكل ال *max−max* ***B.M.D.***

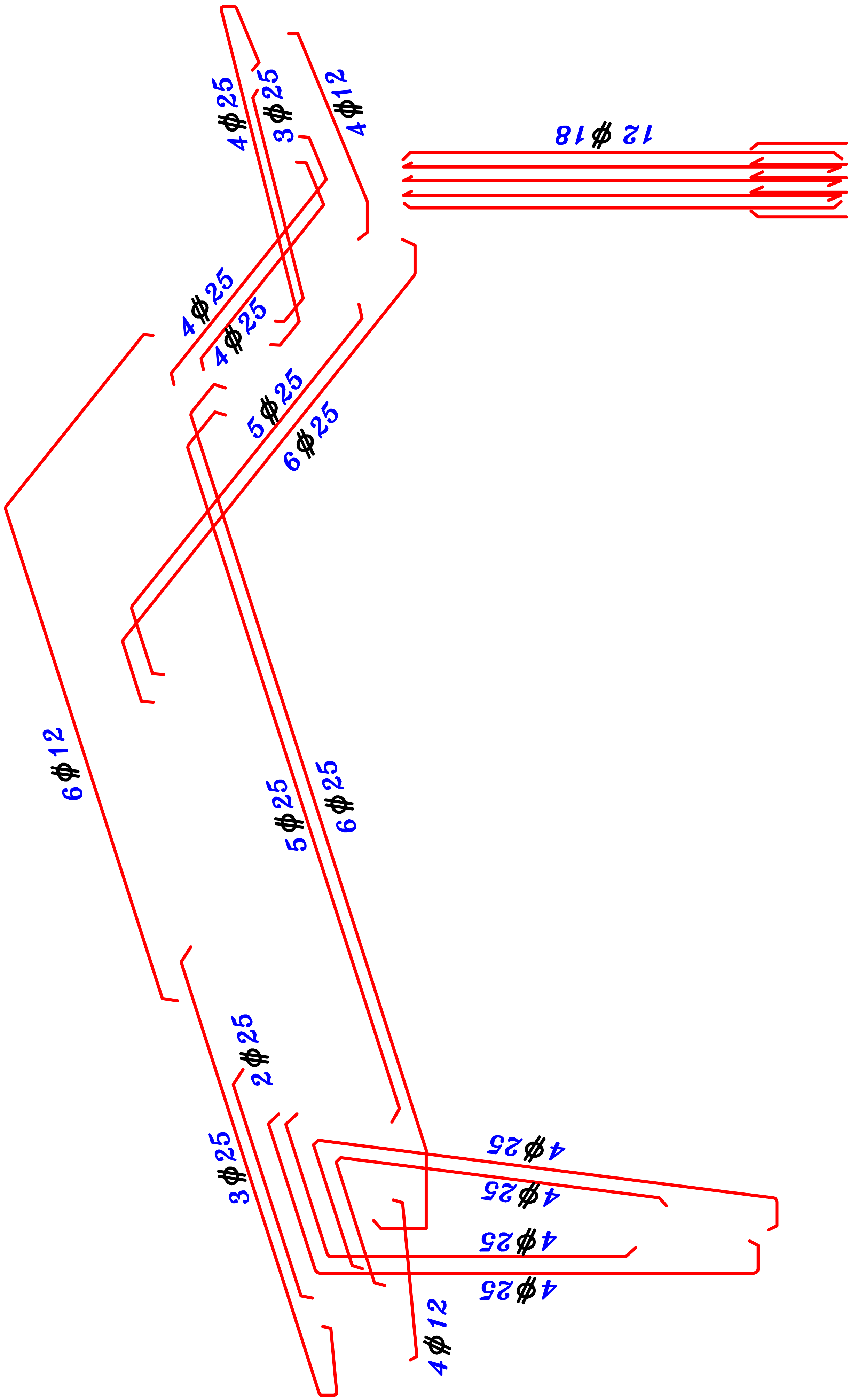
و لكن نرسم التسليح لاى حاله تحميل أولا ثم نكمل التسليح من حاله التحميل الاخرى







تباعد
تباعد
تداخل
تباعد


4φ25
3φ25
4φ12
12φ18
4φ25
4φ25
5φ25
6φ25
6φ12
5φ25
6φ25
2φ25
3φ25
4φ25
4φ25
4φ25
4φ25
4φ12

Example.

Fig.(1) shows concrete dimensions and statical system of a Frame in an industrial building. The interior spans of the Frame are subjected to dead and live loads equal to **30, 15 kN/m`** respectively while the external cantilevers are subjected to total load equal to **35 kN/m`**

It is required to :

1- Estimate the concrete dimensions For all elements of the Frame.

2- Draw the absolute (max-max) Bending Moment Diagram.

3- Draw the Normal Force and Shear Force Diagrams (Case of total load only).

4- Design the critical sections of the Frame For bending and/or normal Force.

5- Check shear stresses at the critical sections.

6- Design column (a-b) as a braced column.

7- Draw the concrete dimensions and the details of reinforcement in elevation and cross-sections to an appropriate scale.

Use: $F_{cu} = 35$ MPa , Steel 36/52 , $b = 350$ mm , Frame spacing $= 5.0$ m

Given Loads are working loads.

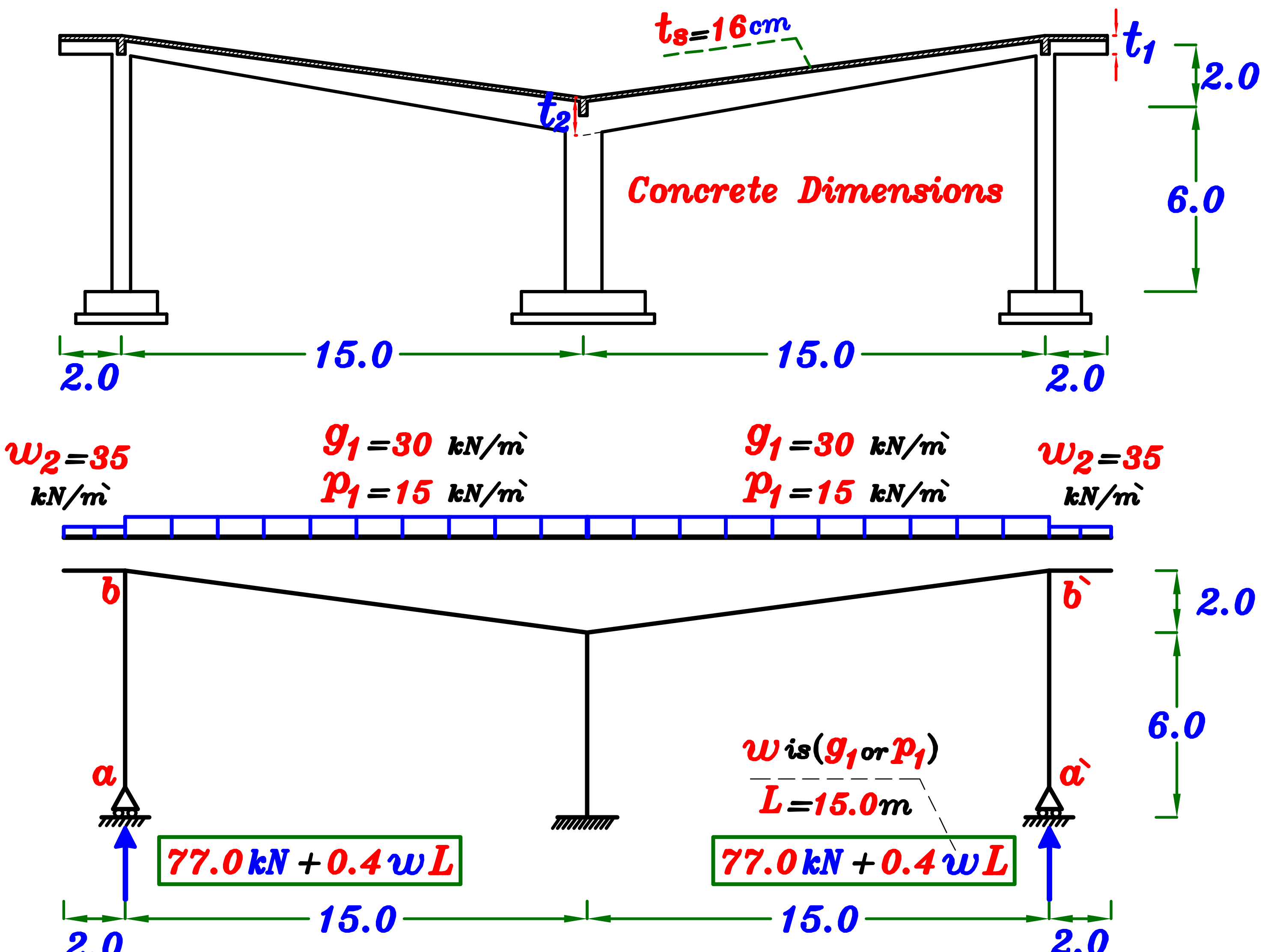
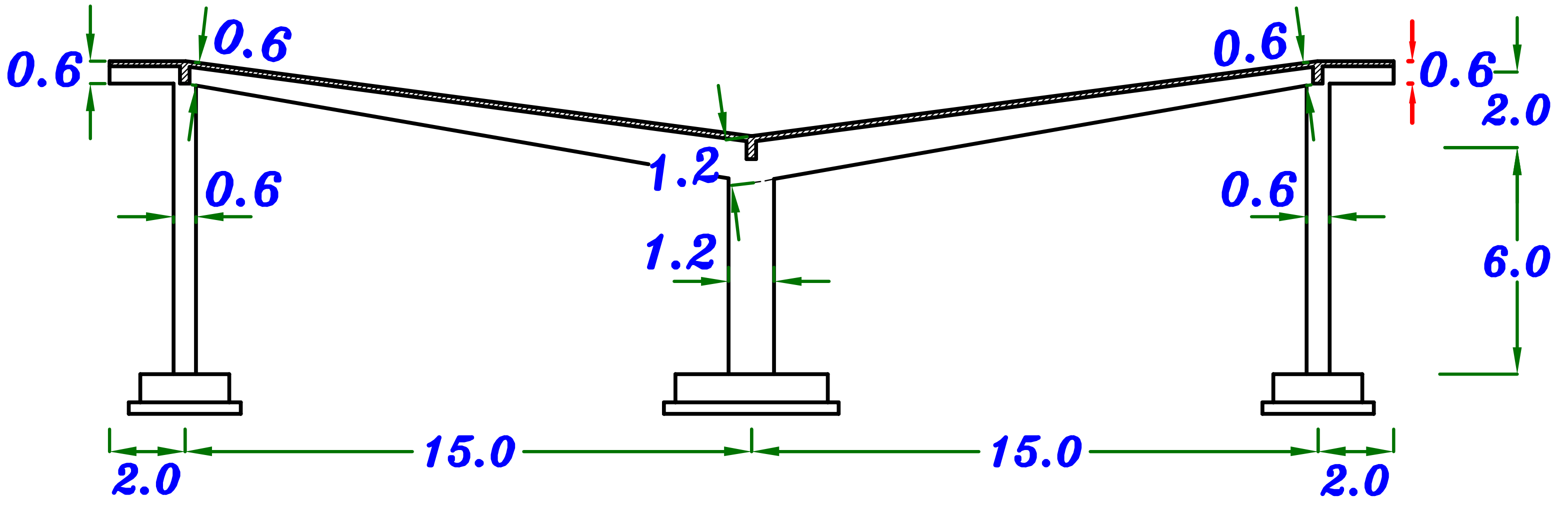
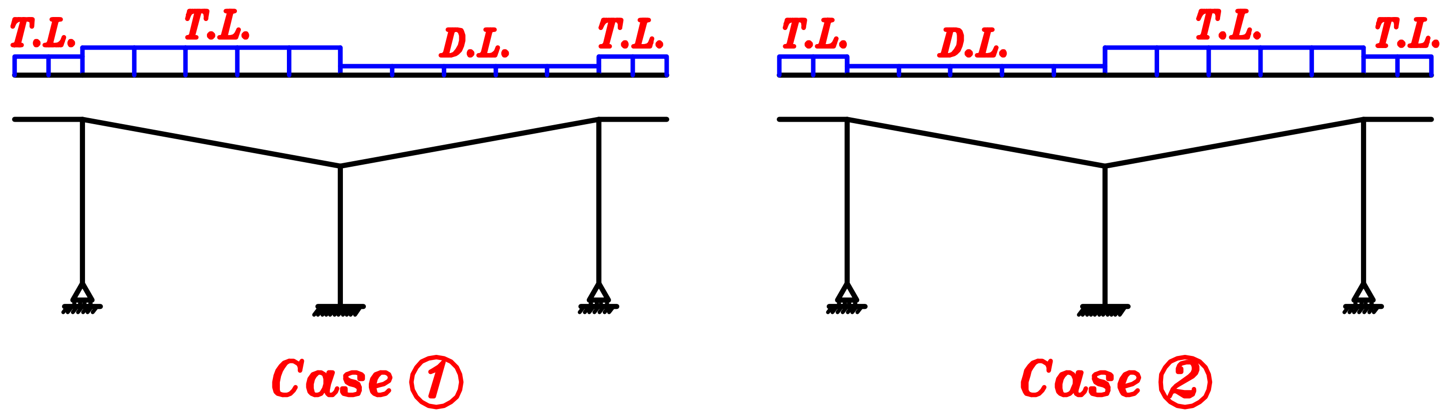


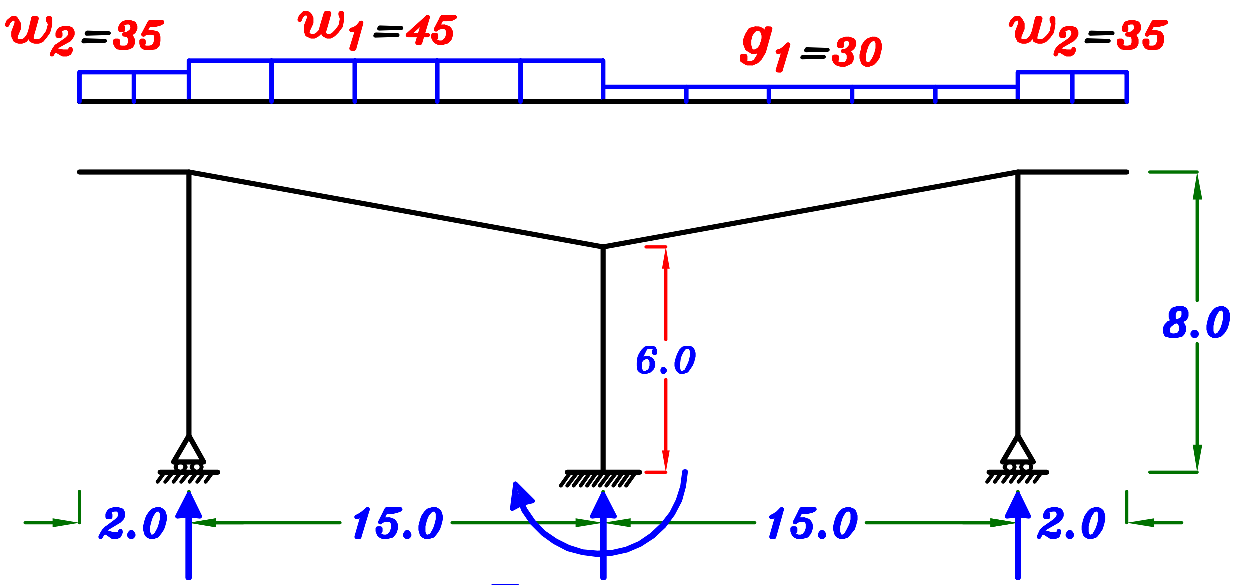
Figure (1)

1- Estimate the concrete dimensions For all elements of the Frame.



2- Draw the absolute (max-max) Bending Moment Diagram.



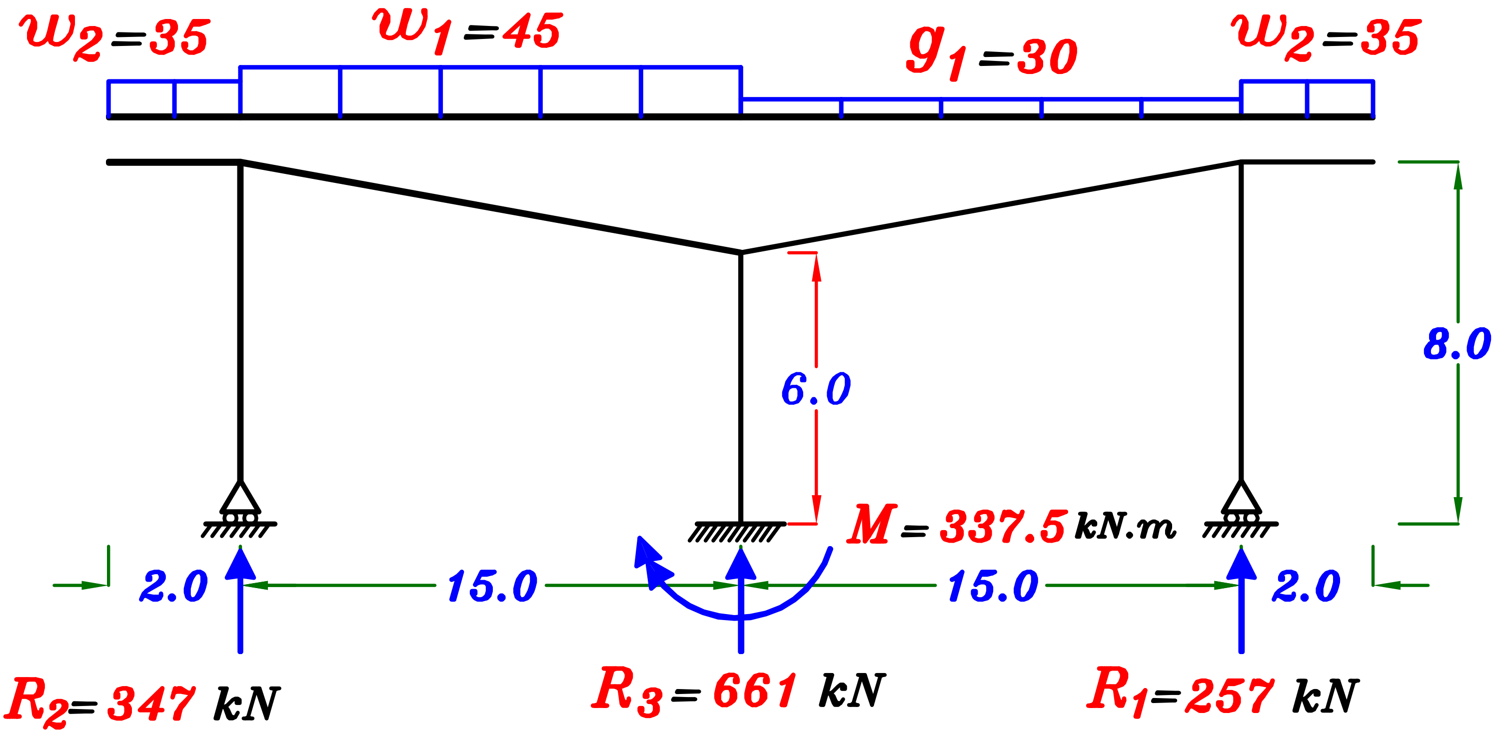


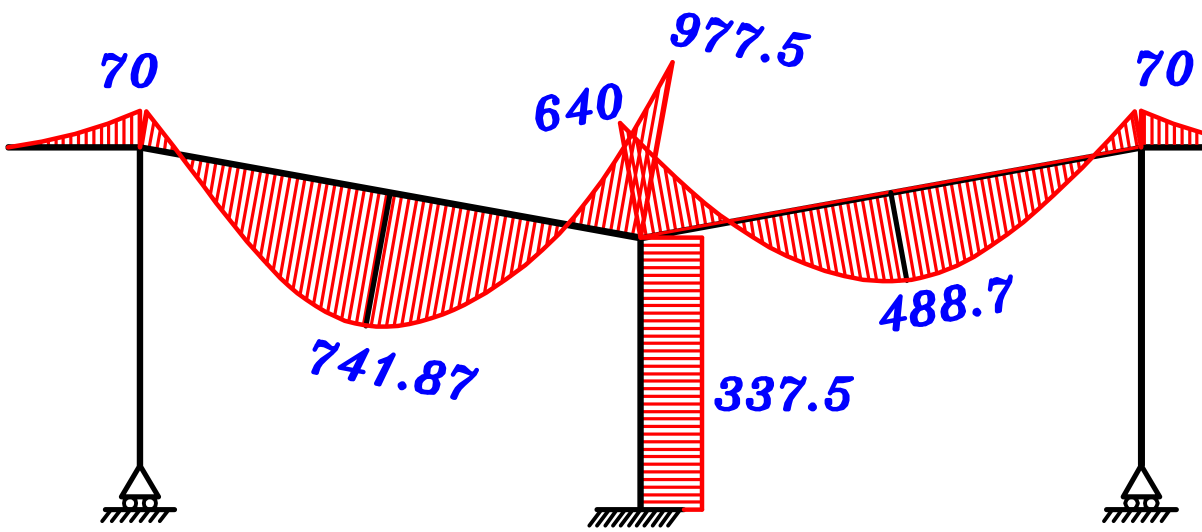
$R_2 = 77 + 0.4\, w_1\, L$
$= 77 + 0.4\,(45)(15)$
$= 347$ kN

$\sum Y =$ Zero

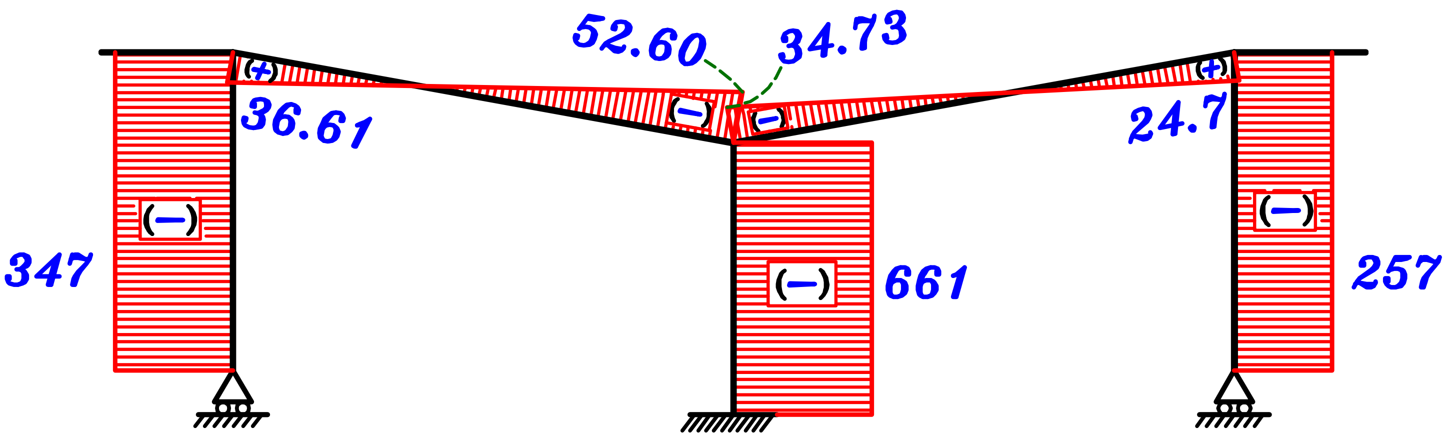
$\boxed{R_3 = 661 \text{ kN}}$

$R_1 = 77 + 0.4\, g_1\, L$
$= 77 + 0.4\,(30)(15)$
$= 257$ kN

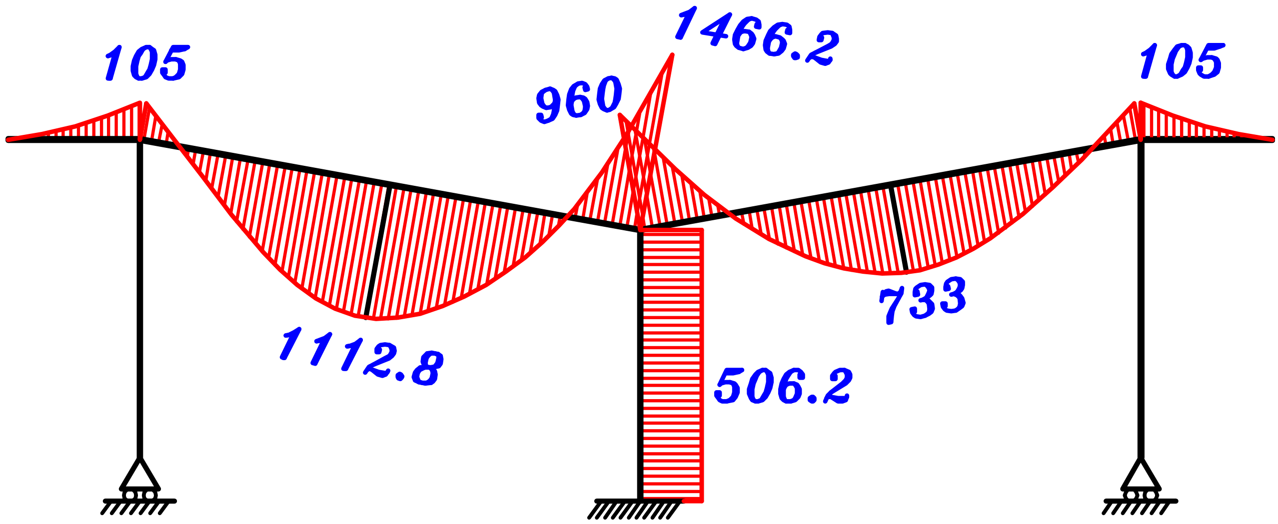




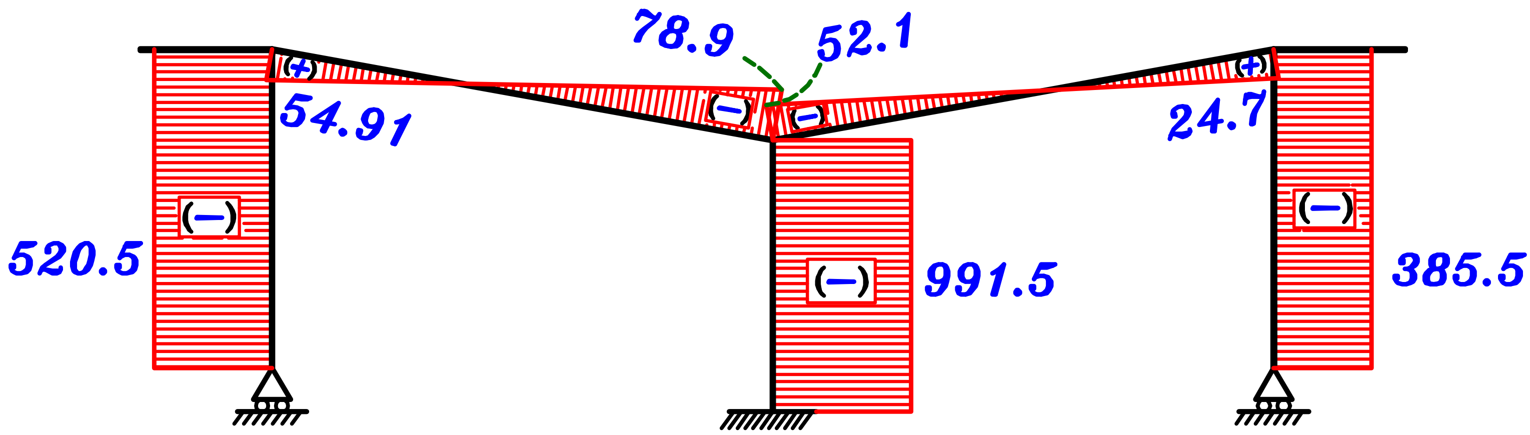
B.M.D. (working)



N.F.D. (working)

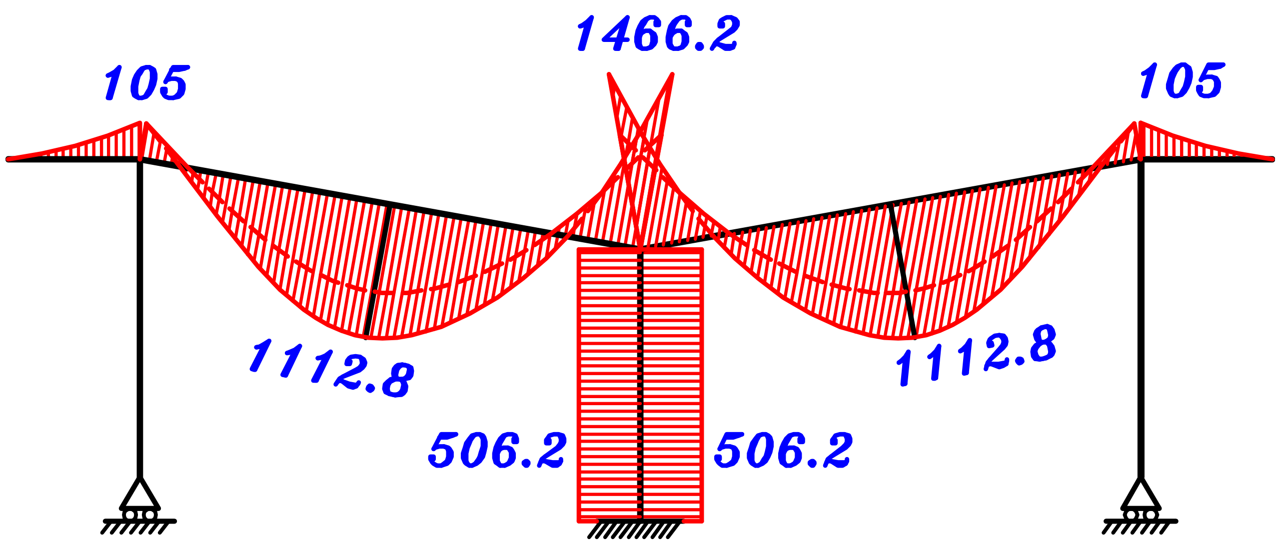


B.M.D. (U.L.)

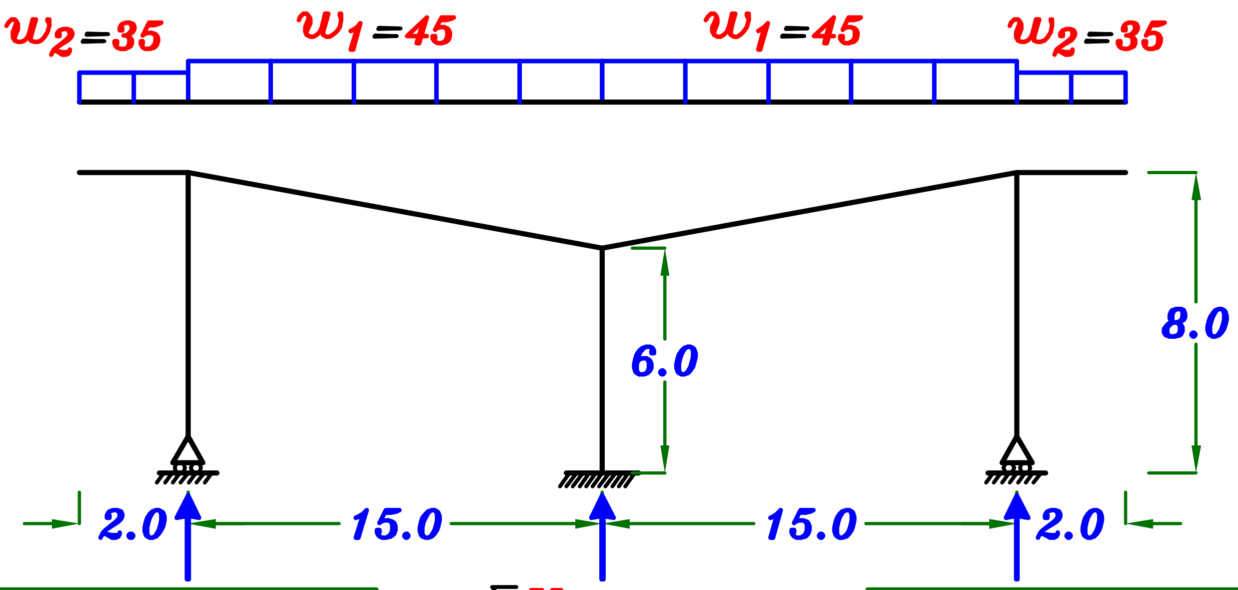


N.F.D. (U.L.)

max–max B.M.D. (U.L.)



3- Draw the Normal Force and Shear Force Diagrams (Case of total load only).

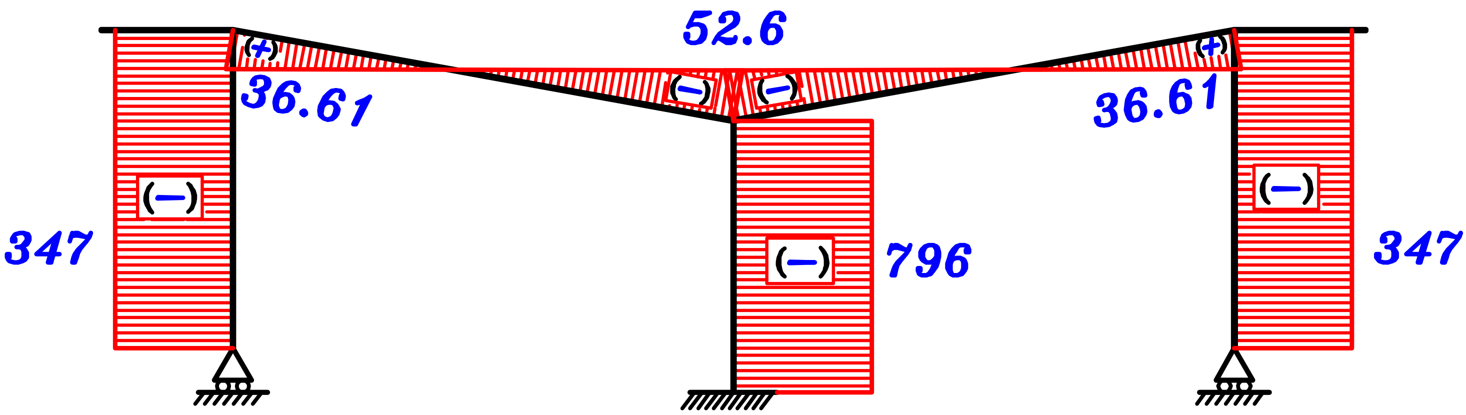


$R_2 = 77 + 0.4\,wL$
$= 77 + 0.4(45)(15)$
$= 347$ kN

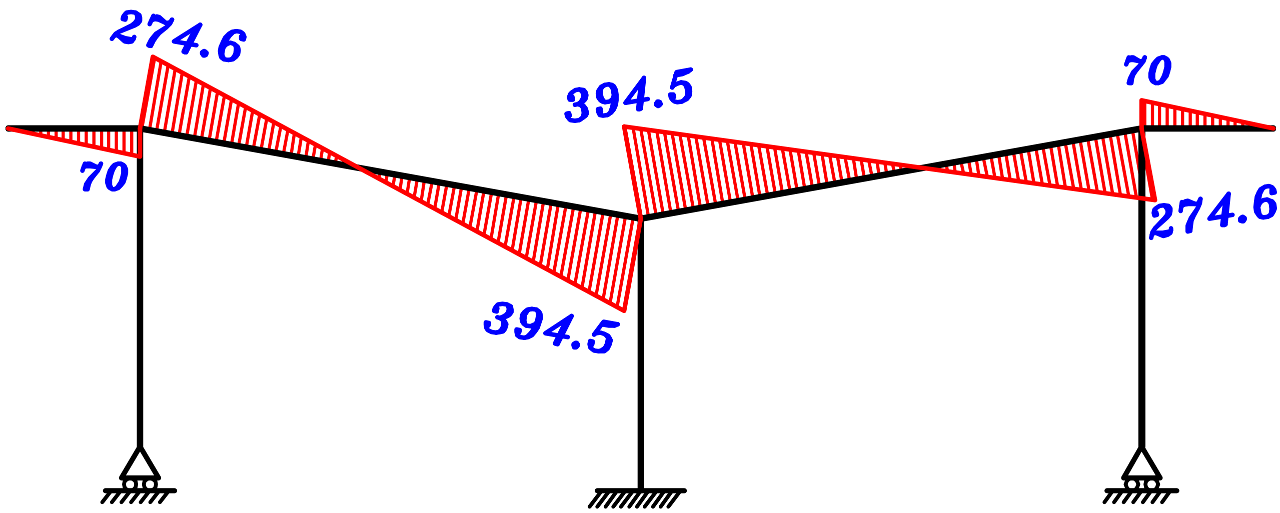
$\sum Y = \text{Zero}$

$R_3 = 796$ kN

$R_1 = 77 + 0.4\,wL$
$= 77 + 0.4(45)(15)$
$= 347$ kN

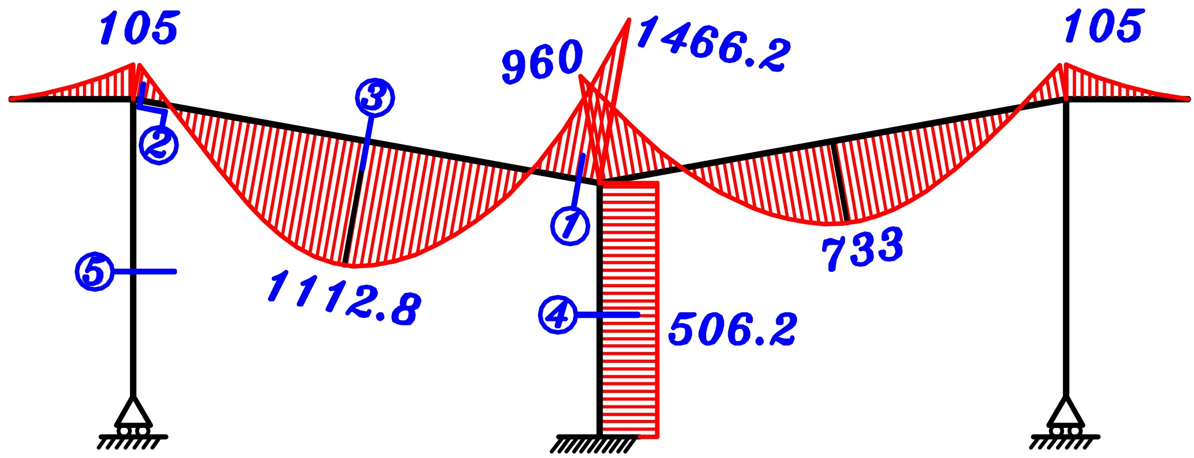


N.F.D. (working)

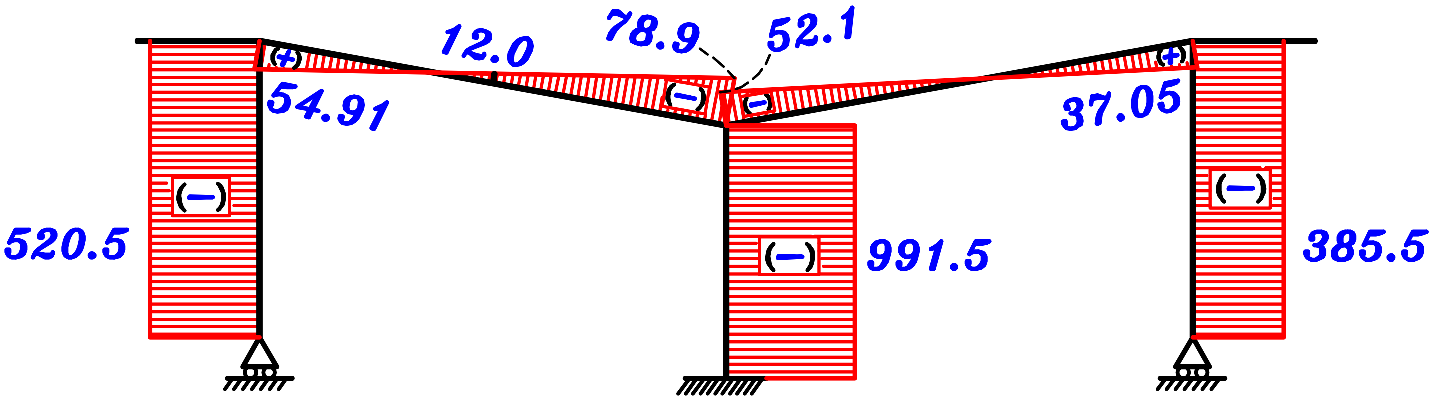


S.F.D. (working)

4- Design the critical sections of the Frame For bending and/or normal Force.



B.M.D. (U.L.)



N.F.D. (U.L.)

Sec. 1 $M = 1466.2$ kN.m , $P = 78.9$ kN , $b = 350$ mm

$$d_o = 3.5\sqrt{\frac{1466.2 * 10^6}{35 * 350}} = 1210.9 \text{ mm} \qquad (\text{as R-Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(1210.9) = (1332 \rightarrow 1574.1) \text{ mm}$$

Take $d = 1400$ mm , $t = 1400 + 100 = 1500$ mm

$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{78.9 * 10^3}{35 * 350 * 1500} = 0.004 < 0.04 \quad \therefore (\text{neglect } P)$$

$\therefore$ Take $d = d_o = 1210.9$ mm

$\therefore$ Take $\boxed{d = 1300 \text{ mm}}$, $\boxed{t = 1400 \text{ mm}}$

$\because \; C_1 = 3.50 \longrightarrow J = 0.78$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{1466.2 * 10^6}{0.780 * 360 * 1210.9} = 4312.1 \; mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 4312.1 \; mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{360}\right) 350 * 1300 = 1682.3 \; mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 4312.1 \; mm^2$ $\quad 9 \phi 25$

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{350 - 25}{25 + 25} = 6.50 = 6.0 \text{ bars}$$

Sec. ② $\quad M = 105 \; kN.m \;, \; T = 54.91 \; kN \;, \; b = 350 \; mm$

$$d_o = 3.5 \sqrt{\frac{105 * 10^6}{35 * 350}} = 324.0 \; mm \quad (\text{as R-Sec.})$$

$$d = (0.9 \rightarrow 1.0)\, d_o = (0.9 \rightarrow 1.0)\,(324.0) = (291.6 \rightarrow 324.0) \; mm < \frac{t}{2}$$

$$t_1 = \frac{t}{2} = \frac{1400}{2} = 700 \qquad \boxed{d = 650 \; mm} \;, \; \boxed{t = 700 \; mm}$$

$$e = \frac{M}{T} = \frac{105}{54.91} = 1.91 \; m \quad \therefore \frac{e}{t} = \frac{1.91}{0.7} = 2.73 > 0.5 \xrightarrow{\text{Use}} e_s$$

$$e_s = e - \frac{t}{2} + c = 1.91 - \frac{0.70}{2} + 0.05 = 1.61 \; m$$

$$M_s = T * e_s = 54.91 * 1.61 = 88.40 \; kN.m$$

$$\therefore 650 = C_1 \sqrt{\frac{88.40 * 10^6}{35 * 350}} \longrightarrow C_1 = 7.65 \longrightarrow J = 0.826$$

$$\because \; A_s = \frac{M_s}{J\,F_y\,d} + \frac{T_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{88.40 * 10^6}{0.826 * 360 * 650} + \frac{54.91 * 10^3}{(360 \backslash 1.15)} = 632.76 \; mm^2$$

<u>Check $A_{s_{min.}}$</u> $\quad A_{s_{req.}} = 632.76 \; mm^2$

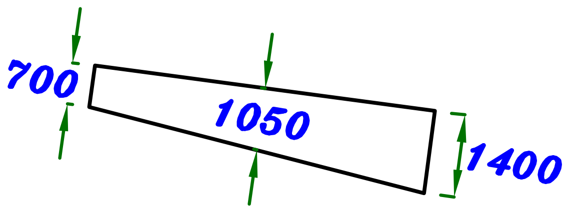
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{360}\right) 350 * 650 = 841.2 \; mm^2$$

$$\therefore \; \mu_{min.}\, b\, d > A_{s_{req.}} \xrightarrow{Use} A_{s_{min.}}$$

$$\left.\begin{array}{l} A_{s_{min.}} = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 650 = 841.2 \; mm^2 \\ 1.3\, A_{s_{req.}} = 1.3 * 632.76 = 822.6 \; mm^2 \end{array}\right\} \text{الأقل} = 822.6 \; mm^2 \quad 2\,\phi\,25$$

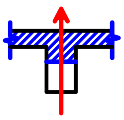
<u>Sec. ③</u> $\quad M = 1112.8 \; kN.m \;,\; P = 12.0 \; kN$

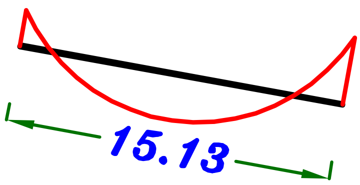
$, \; b = 350 \; mm \quad , \; t = 1050 \; mm$



$$\text{Check} \quad \frac{P}{F_{cu}\, b\, t} = \frac{12.0 * 10^3}{35 * 350 * 1050} = 0.0093 < 0.04 \quad (\text{Neglect } P)$$

The sec. will be T-sec.





$$B = \left\{\begin{array}{l} C.L. - C.L. = Spacing = 5.0 m = 5000 \; mm \\ 16\, t_s + b = 16 * 160 + 350 = 2910 \; mm \\ K\, \frac{L}{5} + b = 0.7 * \frac{15130}{5} + 350 = 2468 \; mm \end{array}\right\}$$

$$\boxed{B = 2468 \; mm}$$

$$\because \; d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, B}} \quad \therefore \; 950 = C_1 \sqrt{\frac{1112.8 * 10^6}{35 * 2468}} \longrightarrow C_1 = 8.36 \longrightarrow J = 0.826$$

$$\therefore A_s = \frac{M_{U.L.}}{J\,F_y\,d} = \frac{1112.8 * 10^6}{0.826 * 360 * 950} = 3939.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 3939.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{35}}{360}\right) 350 * 950 = 1229.4 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3939.2 \text{ mm}^2$ $\quad 9 \phi 25$

Sec. 4 $\quad$ R–Sec. $\quad M = 506.2$ kN.m , $P = 991.5$ kN

$$d_o = 3.5\sqrt{\frac{506.2 * 10^6}{35 * 350}} = 711.5 \text{ mm} \quad (\text{as R–Sec.})$$

$$d = (1.1 \rightarrow 1.3)\, d_o = (1.1 \rightarrow 1.3)\,(711.5) = (782.6 \rightarrow 924.9) \text{ mm}$$

$\therefore$ Take $\boxed{d = 850 \text{ mm}}$, $\boxed{t = 850 + 50 = 900 \text{ mm}}$

$$\because \boxed{t_{(Column)} < 0.8\, t_{(Beam)} \xrightarrow{Take} t_{(Column)} = t_{(Beam)} = 1400 \text{ mm}}$$

Check $\dfrac{P}{F_{cu}\, b\, t} = \dfrac{991.5 * 10^3}{35 * 350 * 1400} = 0.057 > 0.04 \quad \therefore$ (Don't neglect P)

$\therefore$ Design the Sec. on both N.F. & B.M.

$e = \dfrac{M}{P} = \dfrac{506.2}{991.5} = 0.51$ m $\quad \therefore \dfrac{e}{t} = \dfrac{0.51}{1.40} = 0.36 < 0.5 \xrightarrow{Use}$ I.D.

$\therefore$ Use Interaction Diagram

$\zeta = \dfrac{1400 - 200}{1400} = 0.85 = 0.80 \xrightarrow{use}$ ECCS Design Aids Page 4–24

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{991.5 * 10^3}{35 * 350 * 1400} = 0.057 \\[2ex] \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{506.2 * 10^6}{35 * 350 * 1400^2} = 0.02 \end{array}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 35 * 10^{-4} = 3.5 * 10^{-3}$

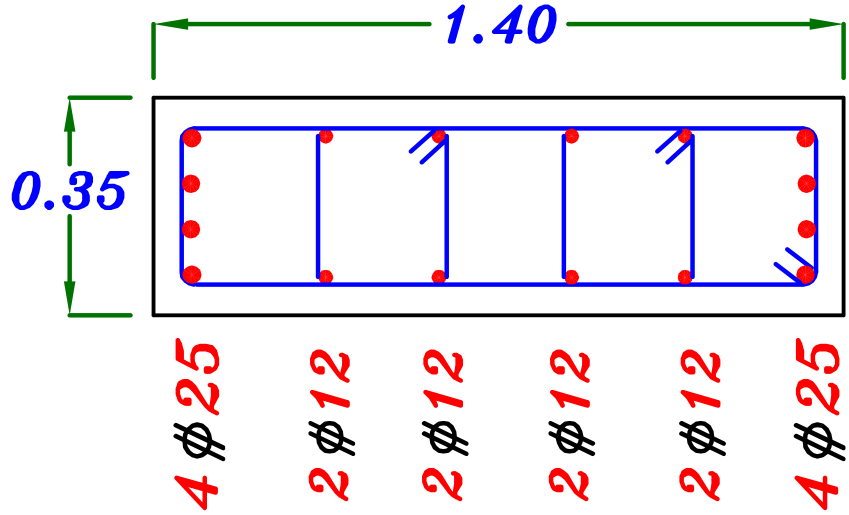
$A_s = A_{s`} = \mu * b * t = 3.5 * 10^{-3} * 350 * 1400 = 1715 \ mm^2$

– Check $A_{s_{min.}} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 350 * 1400 = 3920 \ mm^2$

$A_{s_{Total}} = A_s + A_{s`} = 2 * 1715 = 3430 \ mm^2 \quad \therefore A_{s_{Total}} > A_{s_{min.}}$

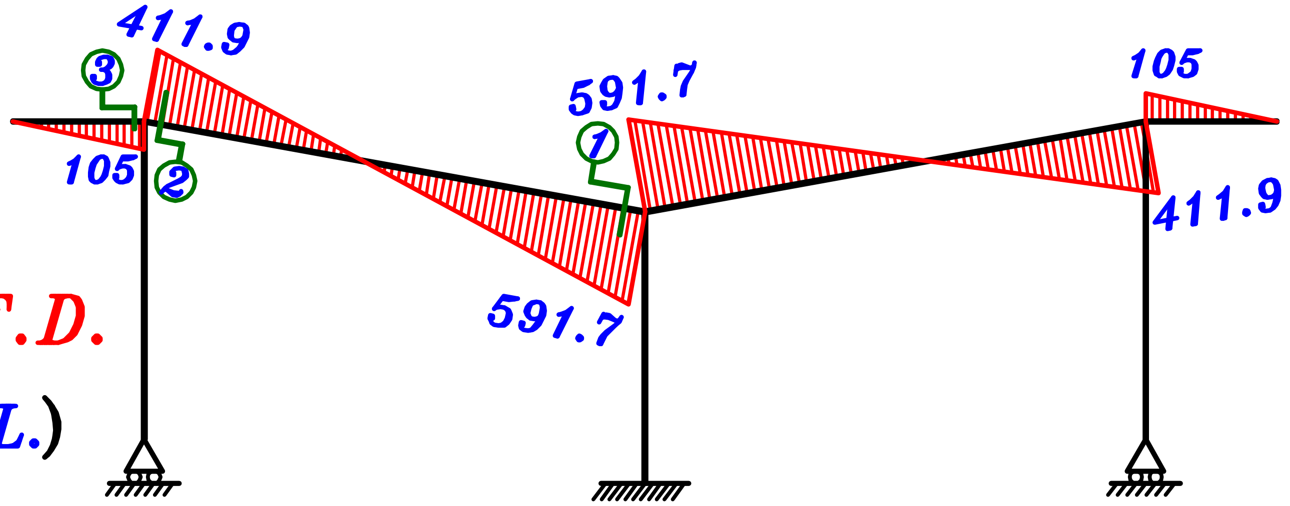
$\therefore$ take $A_s = A_{s`} = \frac{A_{s_{min.}}}{2} = \frac{3920}{2} = 1960 \ mm^2$ $4\phi25$

يجب أن يكون التسليح متساوى فى الجهتين لان العزم من الممكن ان يكون موجود فى ايا من الجهتين .

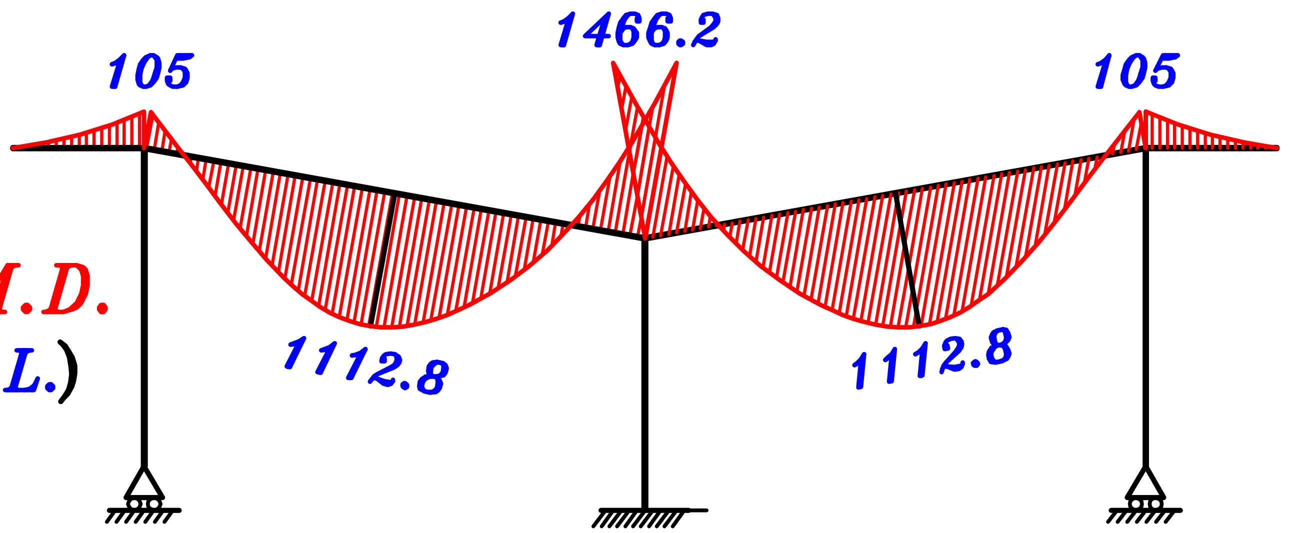


5- Check shear stresses at the critical sections.

S.F.D.
(U.L.)

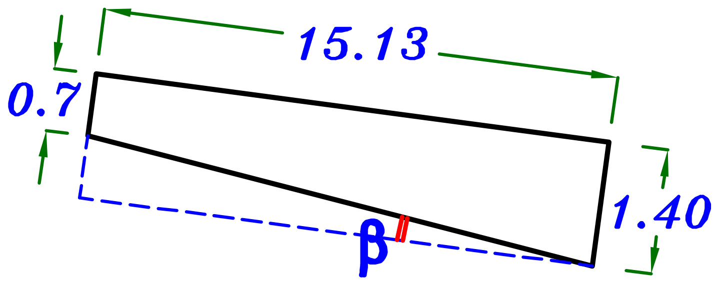


B.M.D.
(U.L.)



Check Shear.

$$\boxed{q_U = \frac{Q}{b\,d} - \frac{M \tan\beta}{b\ d^2}}$$



$$\tan\beta = \frac{0.7}{15.13} = 0.046$$

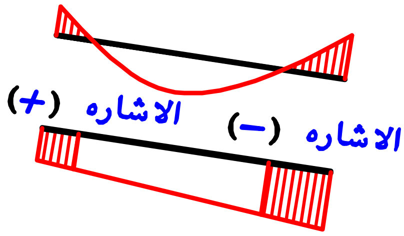
- Allowable shear stress.

- $q_{cu} = 0.24\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.24\sqrt{\frac{35}{1.5}} = 1.16\ N\backslash mm^2$

- $q_{max.} = 0.7\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.7\sqrt{\frac{35}{1.5}} = 3.38\ N\backslash mm^2$

Sec. ① $Q = 591.7\ kN$, $M = 1466.2\ kN.m$

$d = 1300\ mm$



$$q_U = \frac{Q}{b\,d} - \frac{M\tan\beta}{b\,d^2} = \frac{591.7*10^3}{350*1300} - \frac{1466.2*10^6*0.046}{350*1300^2} = 1.186\ N/mm^2$$

$\therefore\ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8 \backslash m'$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n A_s (F_y \backslash \gamma_s)}{b\ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.186 - \frac{1.16}{2} = \frac{2*50.3\ (240\backslash 1.15)}{350*S} \longrightarrow S = 98.9\ mm < 100\ mm$$

* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$$1.186 - \frac{1.16}{2} = \frac{2*78.5\ (240\backslash 1.15)}{350*S} \longrightarrow S = 154.5\ mm > 100\ mm \ \therefore\ o.k.$$

$\therefore$ No. of stirrups$\backslash m' = \frac{1000}{S} = \frac{1000}{154.5} = 6.47 = 7.0$

$\therefore$ Use Stirrups $\boxed{7\phi 10 \backslash m'}$ 2 branches

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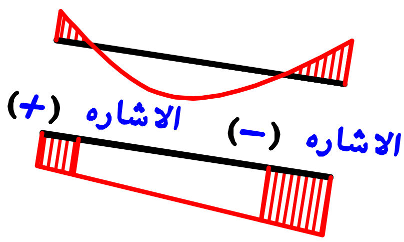
Sec. ② $Q = 411.9\ kN$, $M = 105\ kN.m$

$d = 650\ mm$



$$q_U = \frac{Q}{b\,d} - \frac{M\tan\beta}{b\,d^2} = \frac{411.9*10^3}{350*650} - \frac{105*10^6*0.046}{350*650^2} = 1.78\ N/mm^2$$

$\therefore\ q_{cu} < q_U < q_{max.}$ $\therefore$ We need Stirrups more Than $5\phi 8 \backslash m'$

$\therefore$ Use $q_s = q_u - \frac{q_{cu}}{2} = \frac{n A_s (F_y \backslash \gamma_s)}{b\ S}$

* Take $n = 2$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.78 - \frac{1.16}{2} = \frac{2 * 50.3\ (240\backslash 1.15)}{350 * S} \longrightarrow S = 50.0\ mm < 100\ mm$$

* Take $n = 2$, $\phi\ 10 \longrightarrow A_s = 78.5\ mm^2$

$$1.78 - \frac{1.16}{2} = \frac{2 * 78.5\ (240\backslash 1.15)}{350 * S} \longrightarrow S = 78.15\ mm < 100\ mm$$

* Take $n = 4$, $\phi\ 8 \longrightarrow A_s = 50.3\ mm^2$

$$1.78 - \frac{1.16}{2} = \frac{4 * 50.3\ (240\backslash 1.15)}{350\ \ S} \longrightarrow S = 100.15\ mm > 100\ mm \ \therefore\ o.k.$$

$$\therefore \text{No. of stirrups}\backslash m` = \frac{1000}{S} = \frac{1000}{100.15} = 9.98 = 10.0$$

$\therefore$ Use Stirrups $\boxed{10\phi 8 \backslash m`}$ 4 branches

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Sec. ③

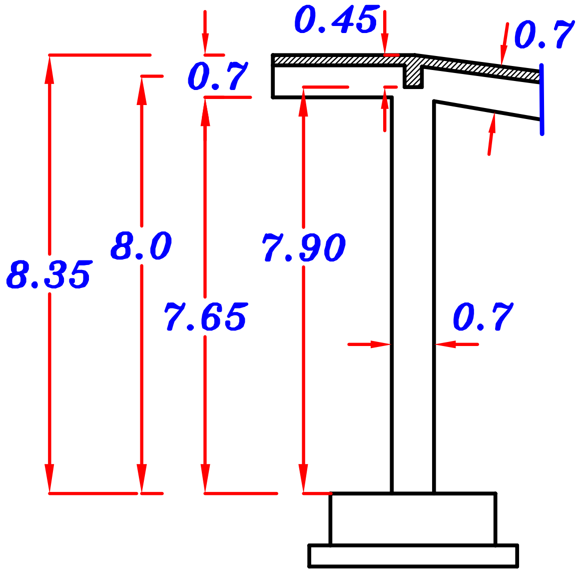
$Q = 105.0\ kN$

$$\therefore\ q_U = \frac{Q}{b\ d} = \frac{105.0 * 10^3}{350 * 650} = 0.461\ N\backslash mm^2$$

$\therefore\ q_U < q_{cu} \longrightarrow$ Use min. stirrups $\boxed{5\phi 8 \backslash m`}$ 2 branches

6-Design column (a-b) as a braced column.

$$\boxed{P_{U.L.} = 520.5 \; kN}$$



① **In plane.** $t = 0.70 \; m$

$$\left.\begin{array}{l} \text{Upper Case } ① \\ \text{Lower Case } ① \end{array}\right\} k = 0.75$$

$$H_o = 7.65 \; m$$

$$\lambda_b = \frac{0.75 * 7.65}{0.70} = 8.19 < 15$$

② **Out of plane.** $t = 0.35 \; m$

$$\left.\begin{array}{l} \text{Upper Case } ① \\ \text{Lower Case } ① \end{array}\right\} k = 0.75$$

$$H_o = 7.90 \; m$$

$$\lambda_b = \frac{0.75 * 7.90}{0.35} = 16.92 > 15 \quad \text{Long column}$$

Take the bigger value of $\lambda_b = 16.92$ (Out of plane)

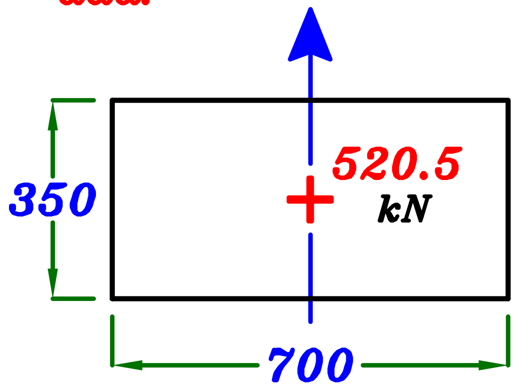
The Buckling Out of plane.

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{16.92^2 * 0.35}{2000} = 0.050 \; m$$

$$M_{add.} = P * \delta = 520.5 * 0.050 = 26.02 \quad kN.m$$

$$e = \delta = \frac{M}{P} = \frac{26.02}{520.5} = 0.05\ m$$

$$M_{add.} = 26.02\ kN.m$$



$$\frac{e}{t} = \frac{0.05}{0.35} = 0.143 < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{0.35-0.1}{0.35} = 0.71 \xrightarrow{Take} \zeta = 0.7 \xrightarrow{Use} I.D. \ \boxed{ECCS\ \ Page\ (4\text{–}25)}$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{520.5 * 10^3}{35 * 700 * 350} = 0.060 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{26.02 * 10^6}{35 * 700 * 350^2} = 0.008 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$As = As` = \mu * b * t = \rho * F_{cu} * 10^{-4} * b * t = 1.0 * 35 * 10^{-4} * 700 * 350 = 857.5\ mm^2$$

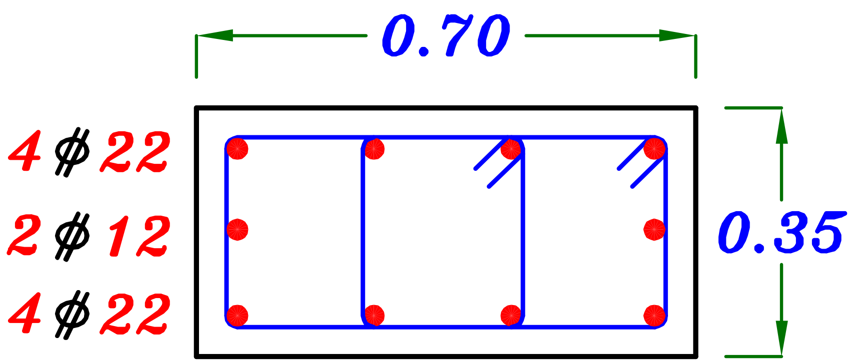
$$As_{total} = As + As` = 1715\ mm^2$$

$$As_{min} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

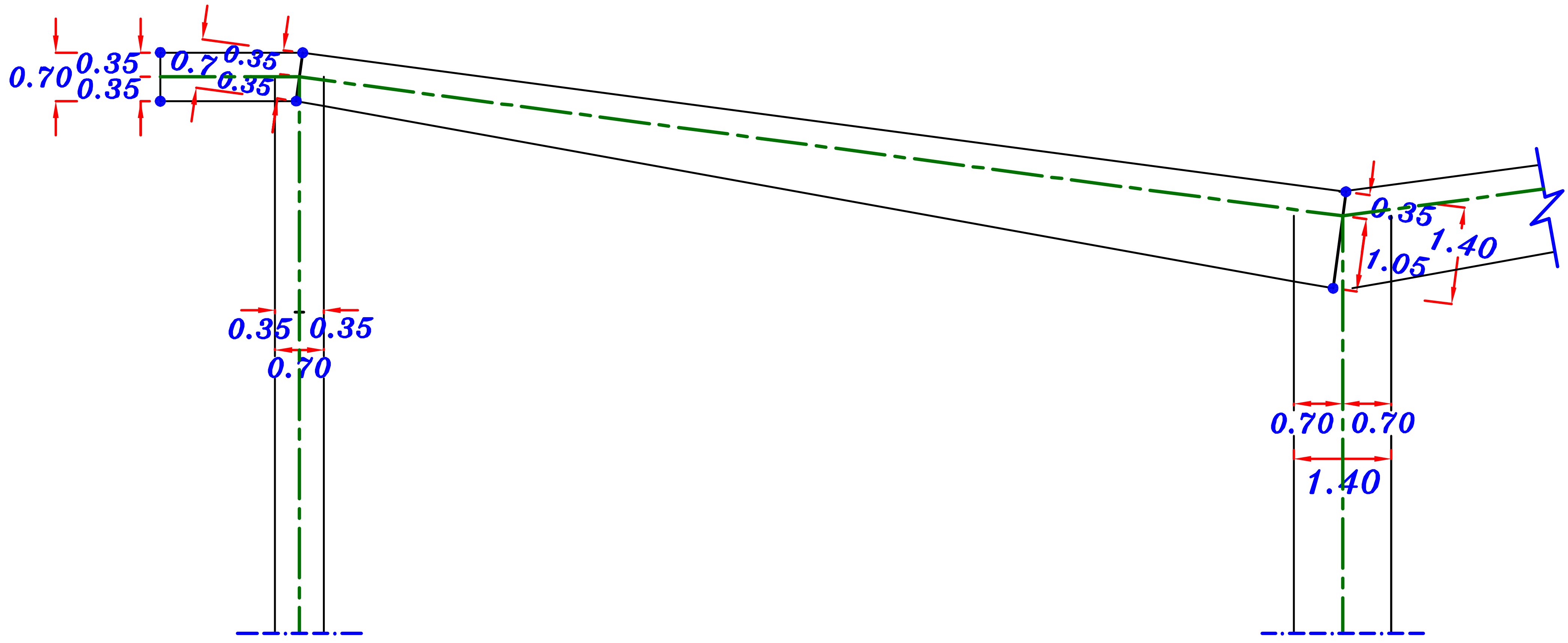
$$= \frac{0.25 + 0.052\,(16.92)}{100} * 700 * 350 = 2768\ mm^2 > As_{total}$$

$$As = As` = \frac{As_{min}}{2} = \frac{2768}{2} = 1384\ mm^2$$

4 ϕ 22

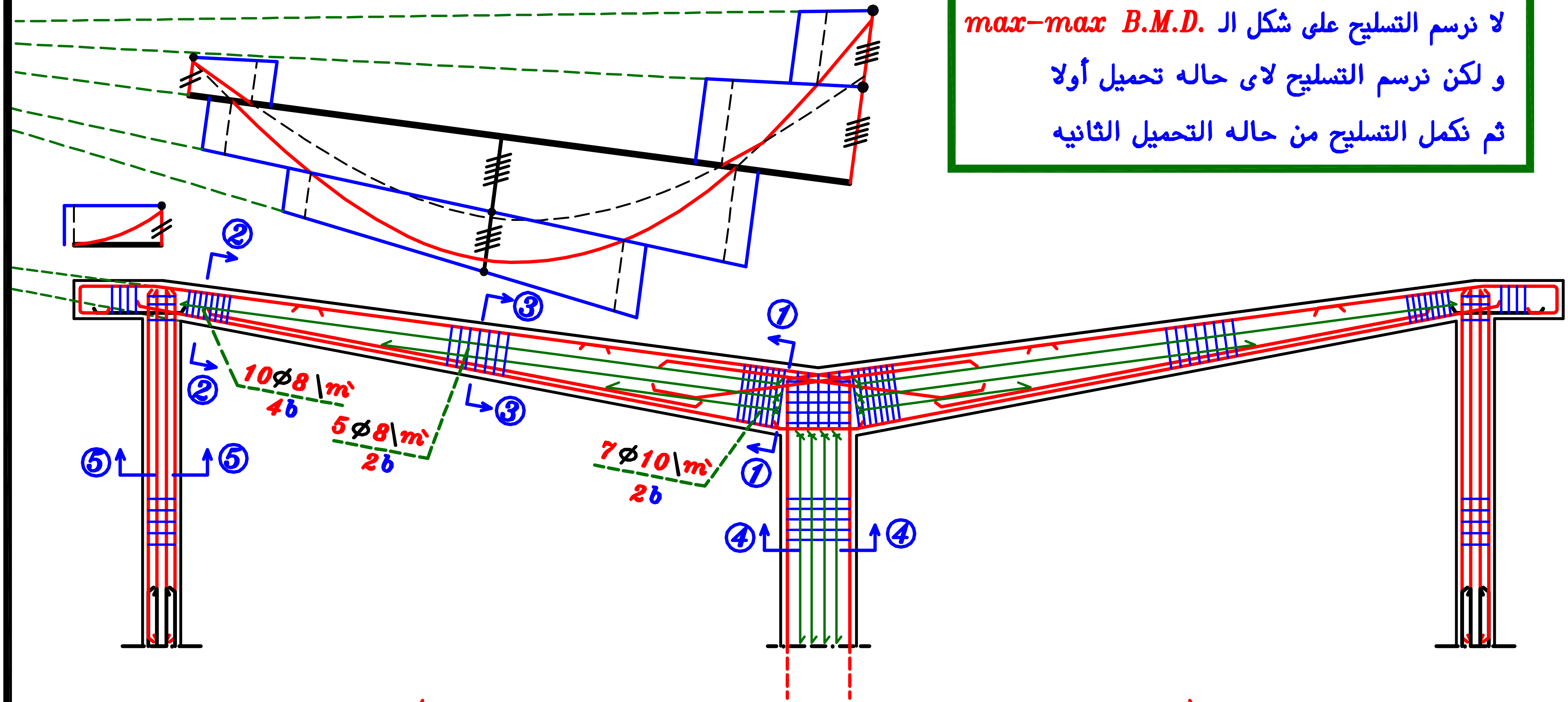


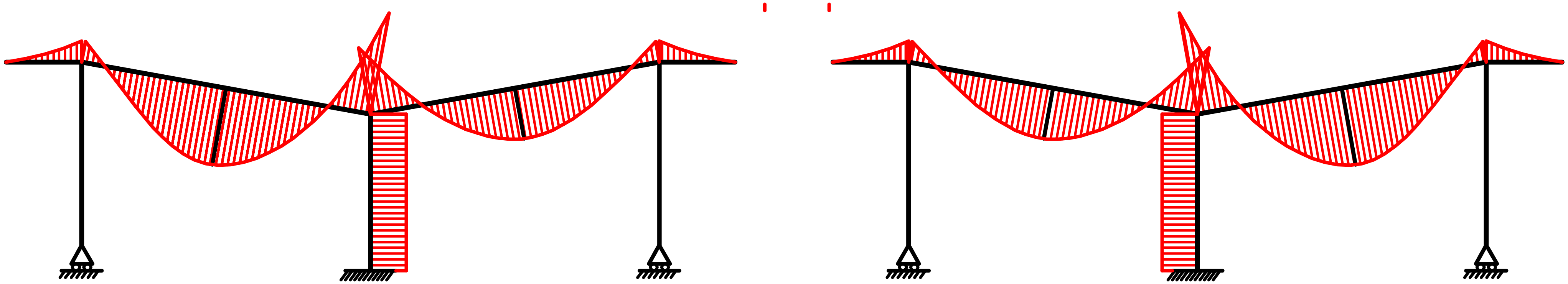
7- Draw the concrete dimensions and the details of reinforcement in elevation and cross-sections to an appropriate scale.



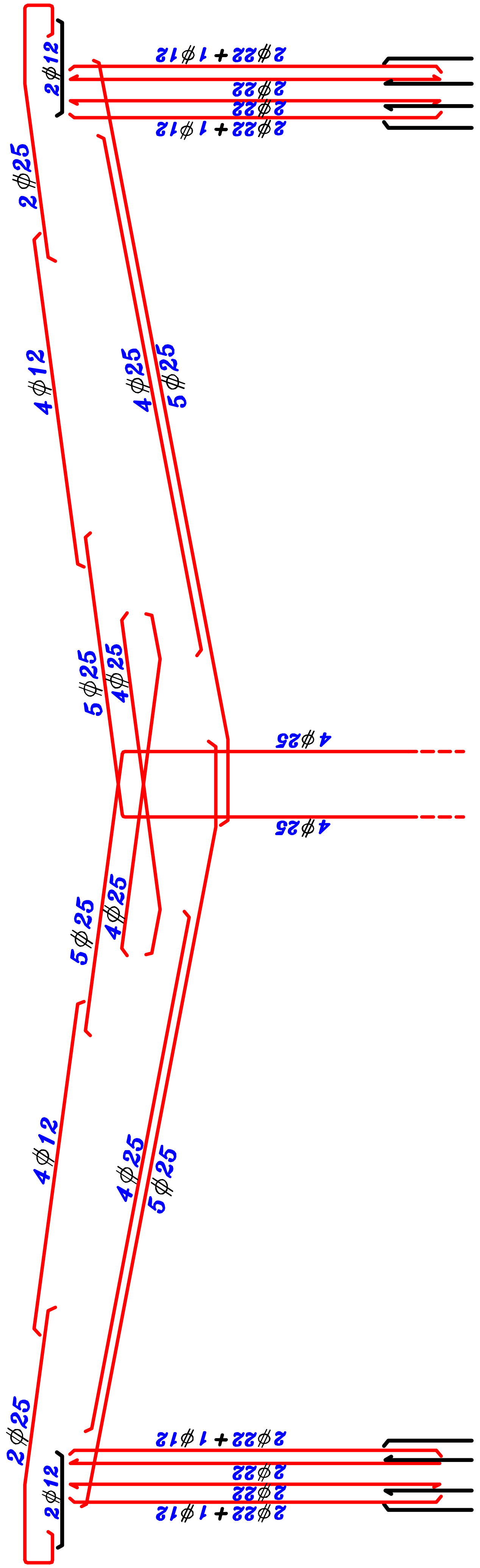
R.F.T. of the Frame.

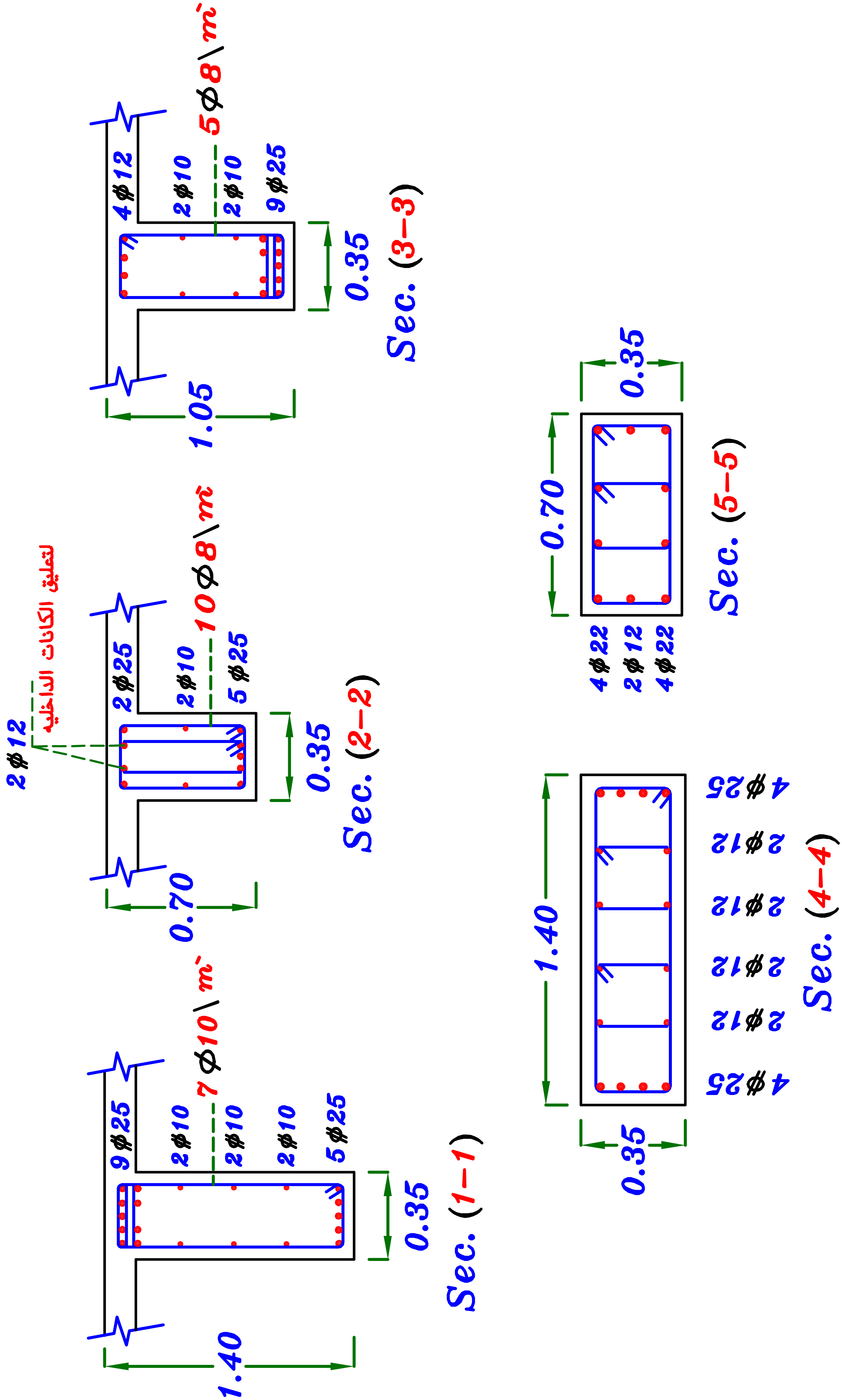
لا نرسم التسليح على شكل الـ max-max B.M.D.
و لكن نرسم التسليح لأى حاله تحميل أولا
ثم نكمل التسليح من حاله التحميل الثانيه





نبدأ بتسليح حاله من الحالتين ثم نزود بالثانيه


2 ϕ12
2 ϕ22 + 1 ϕ12
2 ϕ22
2 ϕ22
2 ϕ22 + 1 ϕ12
2 ϕ25
4 ϕ12
4 ϕ25
5 ϕ25
5 ϕ25
4 ϕ25
4 ϕ25
4 ϕ25
5 ϕ25
4 ϕ25
4 ϕ12
4 ϕ25
5 ϕ25
2 ϕ25
2 ϕ22 + 1 ϕ12
2 ϕ22
2 ϕ22
2 ϕ12
2 ϕ22 + 1 ϕ12


Sec. (1-1)
1.40
0.35
9Φ25
2Φ10
2Φ10
2Φ10
5Φ25
7Φ10\m'
Sec. (2-2)
0.70
0.35
2Φ25
2Φ10
5Φ25
10Φ8\m'
2Φ12
لتعليق الكانات الداخليه
Sec. (3-3)
1.05
0.35
4Φ12
2Φ10
2Φ10
9Φ25
5Φ8\m'
Sec. (4-4)
1.40
0.35
4Φ25
2Φ12
2Φ12
2Φ12
2Φ12
4Φ25
Sec. (5-5)
0.70
0.35
4Φ22
2Φ12
4Φ22